ATTACHMENT D

DAM SAFETY MODIFICATION REPORT (DSMR)
MEMORANDUM FOR RECORD

SUBJECT: Approval of Supplemental Dam Safety Modification Report (SDSMR) and Record of Decision (ROD), Bluestone Dam, New River, Kanawha River Basin, Summers County, West Virginia NIDID: WV00016

1. References:

2. The Great Lakes and Ohio River Division (LRD) Dam Safety Production Center in conjunction with the Huntington District has completed the SDSMR for the Bluestone Dam. The SDSMR was prepared in accordance with ER 1110-2-1156 Safety of Dams – Policy and Procedures dated March 2014 and EC 1165-2-214 Civil Works Review dated December 2012.

3. The SDSMR and associated supporting technical documents have undergone Agency Technical Review, Quality Control and Consistency Review, Independent External Peer Review, and Policy Review. The SFEIS and associated supporting technical documents have undergone Agency Technical Review, Public Review, Independent External Peer Review, and Policy Review. All comments have been satisfied and the necessary revisions incorporated into the SDSMR and SFEIS. Review documentation and required certifications are included in the SDSMR and SFEIS (Appendix P).

4. After reviewing the information provided in References 1.a. and 1.b., including comments from the public and from agencies and responses to comments received, the recommendation of the Dam Safety Senior Oversight Group, and the Dam Safety Officers at the District and Division, we concur with the recommendation identified in the SDSMR and the Record of Decision, with the exception of the construction of a toe drainage gallery (referred to as Phase 6 in the DSMR) as a potential feature to relieve uplift pressures. The SDSMR and accompanying risk assessment demonstrates that sufficient drain efficiency is expected from features recommended under what is referred to as Phase 5. Implementation of additional measures, such as a new toe drainage gallery, would only be warranted by a future risk assessment. Therefore, irrespective of what is in the reference 1.a or 1.b., a new toe gallery (Phase 6) is not included in the approved scope of the Bluestone Dam, DSA Mega-Project.

5. As Phase 6 did not play a significant role in the consideration of defining the affected
environment nor in the subsequent impact analysis reported in the reference 1.b., no revisions to reference 1.a., 1.b. or the Record of Decision are required.

6. Additionally, a new certified Total Project Cost (TPC) has been developed without inclusion of Phase 6 and shall be utilized to develop and manage against a new performance management baseline for the entire project. The TPC report, including the Cost and Schedule Risk Assessment (CSRA), shall be adopted in the project’s Enhanced Mega-Project Project Management Plan. The new TPC Summary is enclosed.

7. While neither reference 1.a nor 1.b require revisions, the accompanying Dam Safety Action Decision Summary (DSADS) and Placemats were revised to match the new TPC that removes Phase 6 from the scope of the project.

8. Finally, the Dam Safety Modification Mandatory Center of Expertise (DSMMCX) and Risk Management Center (RMC) shall ensure that elements included in what is referred to as Phase 5 in the SDSMR are optimized during Preconstruction, Engineering and Design (PED) as further hydraulic modeling is conducted to support design efforts. The completion of these optimization efforts shall be a Headquarters milestone in the project’s Integrated Master Schedule and reported during all future Mega-Project Quarterly In-Progress Reviews until completed.

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DAM SAFETY ASSURANCE MEGA-PROJECT

FINAL DAM SAFETY MODIFICATION REPORT (DSMR)

Supplement to the 1998 Dam Safety Assurance Report

29 June 2017

NID ID: WV08902
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1. EXECUTIVE SUMMARY
Bluestone Dam is a concrete gravity structure located in Summers County near Hinton, West Virginia on the New River, a tributary of the Kanawha River and currently has a Dam Safety Action Classification (DSAC) of 2. The dam was originally constructed in the 1940’s and included flood risk management and power development as its primary authorized purposes (other subsequent authorized purposes are described in Chapter 2). Today it serves as one of the primary means of providing flood risk management benefits to hundreds of thousands of residents and communities downstream of the dam.

1.1. Warranted Continued Federal Interest
The Bluestone Reservoir Project was authorized by the Flood Control Act of June 22, 1936, and June 28, 1938, approving the provisions of Executive Order No. 7183-A, dated September 12, 1935. As indicated above, this act directed the construction of the reservoir as a flood control and hydropower project in accordance with the general plans presented in the supplemental report on the Kanawha River published in House Document No. 91, Seventy-fourth Congress, first session. Additional project authorizations are described in detail in Chapter 2.

The damages prevented by the project through Fiscal Year (FY) 15 are over $2 billion in real dollars (over $5 billion in FY15 dollars). The historic damages prevented from 1947 to 2015 resulted in an average annual benefit of $87 million (FY15 dollars). The population at risk (PAR) for a breach at maximum pool levels is estimated to be approximately 165,000 with associated direct economic damages of over $19 billion (FY15 dollars). Continued Federal investment is warranted to address the current dam safety issues based on the dam’s significant risk to public safety and the substantial flood risk management (FRM) benefits it provides. The proposed modification to the dam will be completed essentially within its existing footprint and will provide the same authorized purposes. Therefore, changes to the existing project purposes or authorization are not warranted at this time.

1.2. Major Modifications Currently Underway
Active construction at Bluestone Dam has been underway since 2000 to address dam safety issues identified in the 1998 Dam Safety Assurance Report (DSA Report). This Dam Safety Modification Report (DSMR) supplements the DSA report, approved in 1998, which focused on addressing the hydrologic deficiency of the dam. Construction features approved in the 1998 DSA report primarily consisted of raising the dam to prevent overtopping, increasing outflow capacity by utilizing the penstocks as an auxiliary spillway, and stabilizing the dam with rock anchors and thrust blocks. Approved modifications began in 2000 and are still ongoing today. The ongoing DSA activities (approved in 1998) are scheduled to continue through 2026. A brief summary of the DSA construction phases are listed in Chapter 2 and additional details can be found in Table 2-2. A figure showing the existing configuration of the dam can be found in Figure 2-4.

There are remaining features that were approved by the 1998 DSA project, but never procured. Those features consist of additional rock anchors (+/- 66) in the dam, completion of a parapet wall across the top of the non-overflow sections, installation of anchors in the apron in the first stage basin, and mitigation commitments. The implementation of these remaining features were reevaluated as part of the Bluestone DSMR. Since the DSA work was approved and awarded, the U.S. Army Corps of Engineers (USACE) moved from a solely standards-based approach for the dam safety program to a dam safety risk informed approach. This study uses the risk informed approach to consider risk reduction to address dam safety issues (risk) not addressed by the DSA project. The primary dam safety issue remaining after the completion of the DSA construction activities is the risk associated with potential failure mode (PFM) 33, spillway monolith instability (A monolith is a large block of concrete that stands on its own and serves as a section of the dam. The dam is made of fifty-five monoliths.).
1.3. Significant Potential Failure Modes

There were multiple risk assessments completed beginning in 2008 and ending in 2016. In the final stages of the risk assessments, three potential failure modes (PFM) were identified as significantly contributing to total risk being above the USACE Tolerable Risk Guidelines (TRG). Subsequently, as part of the existing condition (ECRA) and the future without Federal action condition (FWAC) risk assessments update performed in 2016, only one failure mode, PFM 33 - spillway monolith instability, was considered to contribute significantly to the total incremental risk being above the agency’s tolerance threshold. Therefore, this failure mode is the only one considered actionable and is the primary focus in the development of risk management plans (RMPs) for this DSMS. PFM 33 is described as follows:

Hydrologic event generates significant inflows causing pool to rise to a level which is influenced by gate operability. Uplift acting on a failure plane at or below the structure base increases which could be influenced by flooding of the gallery. Discharges through the primary outlet works reduce passive resistance by flushing out tailwater and scour of the passive rock wedge which could be influenced by the failure of one or more structures within the stilling basin. Two-dimensional driving forces acting on the spillway exceed resisting stabilizing forces and movement of one or more monoliths initiates. Intervention is unsuccessful. This initial movement begins to transfer load to adjacent monoliths and cause deformation of the prestressed anchors. Increased stresses in the anchors induced by this movement cause anchors to begin failing which results in additional excess driving forces which continue to be transferred to adjacent monoliths through additional downstream movement and engaging more monoliths. This process continues until a shallow arch forms across a portion of the dam. Deformation and anchor failure continues until crushing of the concrete or failure in the abutment occurs causing a failure of the arch and multiple monoliths abruptly displace a significant distance downstream leading to breach and loss of pool. (US Army Corps of Engineers, Bluestone Dam Baseline Condition Risk Assessment, 2013)

Two additional failure modes were found to contribute minimally to total incremental risk. These PFMs are PFM 34, Non-overflow Monolith Instability and PFM 35, Abutment Monolith Instability. The risk for both failure modes are driven by overtopping of the dam. The team decided to evaluate these failure modes for further risk reduction opportunities in the context of As Low as Reasonably Practical (ALARP) because of the magnitude of consequences resulting from a dam failure.

1.4. Dam Safety Risk Assessments (ECRA & FWAC)

1.4.1. Existing Condition Risk Assessment (ECRA)

The f/N chart in Figure 1-1 shows that the existing condition risk from PFM 33 and the total risk exceed the tolerable risk limit for life safety as described in ER 1110-2-1156 with the best estimate being more than an order of magnitude greater than the threshold limit. Although PFM 33 contributes over 95% of the total risk, PFM’s 34 and 35 are estimated to be in the “Low Probability – High Consequence” zone for the existing condition. This chart includes the loading and response associated with all gates operating, but also assumes 3 spillway crest gates failing to open to capture the risks associated with the current reliability of the aging operating equipment of the gate. The sensitivity to loading and consequence boxes are only associated with the all gates operate scenario. Although not included, the sensitivity boxes for the 3 gate scenario are expected to shift similarly.

1.4.2. Future without Federal Action Condition (FWAC) Risk Assessment

The FWAC risk assessment (Figure 1-2) differs slightly from the ECRA with primary differences being the assumption of the operation of the dam with respect to the threshold discharge and the mobilization rates for the downstream public in the event of a significant flood. The same methodology and event tree were
used in DAMRAE to calculate a risk estimate for the FWAC. The FWAC chart only assumes the loading and response associated with all gates operating due to the assumption that the gates will be rehabilitated. Table 1-1 shows a direct comparison of the assumptions used in the existing and future risk assessments.

<table>
<thead>
<tr>
<th>Assumptions</th>
<th>Existing Condition (ECRA)</th>
<th>Future Condition (FWAC)</th>
<th>Basis of Change</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hydrology</td>
<td>Overtopping Frequency = 1 in 100,000 to 1 in 10,000,000 Overtopping Frequency Best Estimate = 1 in 1,000,000 Estimated Depth of Overtopping at IDF = 18-ft</td>
<td>At top of dam there is no significant change between ECRA and FWAC condition for the inflow design flood (IDF) assumptions</td>
<td></td>
</tr>
<tr>
<td>Mobilization Rates for New and Kanawha Rivers (Best Estimates)</td>
<td>8 hour = 85% 24 hour = 90%</td>
<td>8 hour = 93% 24 hour = 96%</td>
<td>Given discussions with local and state emergency managers, they expect to implement a reasonable level of improvement in technology; emergency planning and response; and warning system capabilities between the ECRA and FWAC.</td>
</tr>
<tr>
<td>Water Control Manual and Dam Operations</td>
<td>Used original WCM dated 1994 with penstock gates operating after spillway gates are fully opened and prior to dam overtopping.</td>
<td>The revised WCM with a permanent threshold discharge in the primary stilling basin and includes penstock gates operating prior to spillway gates being fully opened.</td>
<td>Due to instability of the stilling basin weir at discharges above 140,000 cfs (threshold discharge), there is higher dam safety risk when the discharge exceeds this threshold. Therefore, it is implied there are less dam safety risk when the discharge is released through the penstock gates.</td>
</tr>
<tr>
<td>Population at Risk (PAR) from failure for the Inflow Design Flood (IDF)</td>
<td>174,002</td>
<td>165,302</td>
<td>The ECRA used the 2014 Census data and the FWAC used population projections from the West Virginia University Bureau of Business and Economic Research. The future populations at risk is projected to decrease over the period of analysis.</td>
</tr>
<tr>
<td>DSA Approved Anchors</td>
<td>66 anchors in addition to what has been procured in Phase 4 DSA construction.</td>
<td>66 anchors in addition to what has been procured in Phase 4 DSA construction.</td>
<td>There is no change between these two assumptions. These anchors are required to realize the ECRA and FWAC risk results. The implementation cost are paid for by Construction General (CG) as opposed to O&amp;M funds.</td>
</tr>
<tr>
<td>Operation and Maintenance of Spillway Crest Gates (Reliability)</td>
<td>Replacement of all hoisting wire ropes and the right-angle gear reducers of gates number 7 and 9.</td>
<td>Assumes gate machinery will be rehabilitated and maintained to ensure reliable operation using O&amp;M program funding.</td>
<td>Responsible dam ownership duties to keep up with operation and maintenance activities.</td>
</tr>
</tbody>
</table>
Figure 1-1: ECRA f-N Chart
Societal tolerable risk limit for average annual life loss

Risks are unacceptable, except in extraordinary circumstances

Lower risks to a tolerable level informed by the ALARP considerations

Figure 1-2: FWAC f-N Chart
1.5. Formulation of Risk Management Plans

The formulation of RMPs focused on the study objective to reduce the incremental risk to below tolerable risk guidelines. Plan formulation began with the brainstorming of risk management measures to address the incremental risks for each individual failure mode. Structural and non-structural measures were developed to address risk. The plans were formulated using the risk management measures as building blocks, mixing and matching features, with the intent of meeting the study objectives and taking advantage of the opportunities.

The final array of plans includes the five required alternatives specified in Engineering Regulation (ER) 1110-2-1156. The five required alternatives are: No Action (FWAC) alternative; meeting full tolerable risk guidelines using ALARP considerations to include applicable essential USACE guidelines; achieving only tolerable risk limit for life-safety; remove structure; and replace structure. The plans were developed using the five required plans as the formulation strategy. A list of the final array is listed below.

1.5.1. Risk Management Plan 1 - No Action (FWAC)

This plan consists of the existing features that are in place today along with routine annual operation and maintenance activities, completion of Phase 3 and 4 of the DSA construction project, and the installation of the +/- 66 anchors. The primary differences between the assumptions in the existing condition risk assessment and the No Action (FWAC) plan are the following:

- Flows from the dam are restricted to 140,000 cfs;
- Slight decrease in population at risk;
- Higher success rate in mobilization; and
- Gate machinery will be rehabilitated.

1.5.2. Risk Management Plan 2 - Dam Removal

This plan includes removal of all or a portion of the Bluestone Dam to eliminate the impoundment. Flows would return to pre-dam conditions eliminating the ability to meet its originally authorized purpose. Flood risk management benefits, which are estimated at greater than $87M annually (FY15 dollars), would no longer be realized. It is expected that the sediment deposited within the reservoir area would be released in whole or in part causing significant impact to downstream Resource Category 1 habitat (recently renamed as “High Value Habitat”, but will be referred to as “Resource Category 1” in this document). Mitigation measures to minimize these effects are part of this plan. This plan also includes improved public education regarding the risk of flooding due to the loss of this dam. The improved educational tools will be developed and implemented in cooperation with local stakeholders and government bodies, as well as state and federal emergency management agencies.

Mitigation requirements as determined the Supplemental Final Environmental Impact Statement (SFEIS) will be part of this plan.

1.5.3. Risk Management Plan 3 - Dam Replacement

This alternative consists of removing the existing dam and designing a new dam to meet all applicable essential USACE guidelines. The replacement dam would be a concrete structure with a spillway, similar to the original design. The replacement would be expected to address all failure modes and meet all tolerable risk guidelines. This plan would also include any mitigation required for impacts to the environment. This plan also includes the development of improved public education regarding the risk of flooding during construction and following construction as even the new dam is unlikely to eliminate all flood hazards. The improved educational tools will be developed and implemented in cooperation with local stakeholders and government bodies, as well as state and federal emergency management agencies.
Mitigation requirements as determined the SFEIS will be part of this plan.

1.5.4. Risk Management Plan 4 - Downstream Conventional Stilling Basin

This plan does not address floods that overtop the dam (Inflow Design Flood (IDF)). This plan includes the construction of a new stilling basin with baffles downstream of the existing stilling basin (For details see Figure 5-6 through Figure 5-8). This new stilling basin extends approximately 300 feet downstream of the end sill of the existing stilling weir apron to the downstream end of the new basin. The existing baffles and end sill of the spillway apron are removed with this plan. The existing weir and weir apron are also removed and replaced with a transition zone between the toe of dam at EL 1,368-ft and the new, deeper stilling basin at EL 1,345-ft, requiring large quantities of excavation of rock. The entire length of the channel chute and transition zone are covered with concrete, and baffle blocks are constructed near the end of the new basin along with appropriate uplift relief drainage features. This plan also consists of a new stilling basin wall constructed to EL 1,425-ft along the right and left bank of the new stilling basin.

This plan also includes the development of improved public education regarding the risk of living downstream of a dam. The development and implementation of these tools will be in cooperation with local stakeholders and government bodies, as well as state and federal emergency management agencies. Since this plan is not designed for floods that overtop the dam, waterproofing of openings are part of this plan to prevent flooding of the drainage galleries.

In addition, the approximate 66 anchors of the FWAC measures that were approved in the 1998 DSA report, but never procured, will be installed. Construction of these anchors was included in the ECRA and FWAC risk assessments and will be implemented with construction general (CG) funds. The anchors will be designed and procured with the dam safety modification (DSM) project to ensure compatibility with the proposed design. The remote operation of the crest gates will be implemented due to the risk reduction gained for life safety risk to dam personnel.

Mitigation requirements as determined the SFEIS will be part of this plan.

1.5.5. Risk Management Plan 5 - Transitional Flip Stilling Basin

This plan does not address floods that overtop the dam (IDF). This alternative plan would include the construction of a new concrete apron slab within the existing stilling basin with foundation anchors and appropriate uplift relief drainage features that would transition to a flip bucket spillway just upstream of the existing weir (See Figure 5-9 through Figure 5-11). Discharges up to a certain threshold, approximately half the design discharge, will result in a hydraulic jump upstream of the flip bucket due to the sloping apron. Greater discharges will result in the formation of a flip that will transmit discharge downstream of the basin. Often a flip basin spillway would be constructed with an adjacent downstream plunge pool of adequate depth to allow the water to fall into downstream waters without creating a large scour hole. If plunge pool construction were included in this alternative plan, such a pool would impact additional Resource Category 1 habitat. If this plan does not include an adjacent plunge pool, it would include the construction of a concrete cutoff wall downstream of the existing stilling weir apron to prevent possible scour caused by the plunging water from migrating upstream and compromising the stability of the dam. Since RMP 5 is not designed for floods that overtop the dam, waterproofing of openings is included in this plan to prevent flooding of the drainage galleries.

This plan also includes the development of improved public education regarding the risk of living downstream of a dam. This would be developed and implemented in cooperation with local stakeholders and government bodies, as well as state and Federal emergency management agencies.

Mitigation requirements as determined the SFEIS will be part of this plan.
In addition, the stilling weir will be stabilized and the approximate 66 anchors will be installed. The anchors are part of the FWAC condition, but will be implemented with construction general (CG) funds because they are required to meet the estimated ECRA and FWAC. The anchors will be designed and procured with the dam safety modification project to ensure compatibility. The remote operation of the crest gates will be implemented due to the risk reduction gained for life safety risk to dam personnel.

1.5.6. Risk Management Plan 6/Tentatively Selected Plan (TSP) - Stilling Basin with Super-cavitating Baffles with lower parapet wall

This RMP does not address floods that overtop the dam (IDF). This plan consists of various features and risk management measures formulated to ensure stability of the stilling basin and the dam during extreme flood events. Under this plan, the modified stilling basin would remain a two-stage system within the same footprint with the following modifications and features (see Figure 5-12 through Figure 5-15 for additional details):

- Demolition of the existing first stage baffle blocks and construction of new, larger super-cavitating baffle blocks with anchors immediately downstream of the existing apron.
- A protective concrete apron overlay for the natural riverbed in the first stage basin between the new baffle block monoliths and the existing stilling weir with an system of underdrains.
- Construction of a permanent divider wall to bisect the stilling basin with a new gallery plumbed to the underdrains in the apron.
- Anchors in both the existing and new concrete slabs (apron) to stabilize against uplift pressures in the foundation created by seepage from the reservoir.
- Construction of a new drainage gallery (if determined necessary) within the dam to relieve some of the uplift pressures.
- Stabilizing the left and right training walls with anchors.
- Placement of scour protection behind the left and right training walls.
- Installation of stabilization anchors in the stilling weir.
- Demolition/reconstruction of the second stage end sill and baffle blocks within their existing footprint and anchoring the apron to ensure stability and satisfactory performance.
- Completion of the partially constructed parapet wall in the non-overflow section to reduce the probability of overtopping (as an ALARP consideration).

RMP 6 does not meet all essential USACE guidelines because it does not address the full IDF. Therefore, waterproofing of openings is included in this plan to prevent flooding of the drainage galleries.

This plan also includes the development of improved public education regarding the risk of living downstream of a dam. This would be developed and implemented in cooperation with local stakeholders and government bodies, as well as state and Federal emergency management agencies.

Mitigation requirements as determined the SFEIS will be part of this plan.

In addition, the stilling weir will be stabilized and the approximate 66 anchors will be installed. The anchors are part of the FWAC condition, but will be implemented with construction general (CG) funds because they are required to meet the estimated ECRA and FWAC. The anchors will be designed and procured with the dam safety modification (DSM) project to ensure compatibility. The remote operation of the crest gates will be implemented due to the risk reduction gained for life safety risk to dam personnel.
1.5.7. Risk Management Plan 7 - Concrete Overlay in Stilling Basin

This plan targets achieving only the tolerable risk limit for life safety only. This plan is very similar to RMP 6 except it doesn’t include the super-cavitating baffles or the training wall improvements. This alternative includes a protective concrete apron overlay for the approximately 180 feet of natural riverbed in the first stage between the apron and the existing stilling weir. To further stabilize the foundation against pressure created by seepage from the reservoir pool, this alternative would include anchors in both the existing apron and new concrete slabs (See Figure 5-16 and Figure 5-17). This plan was developed to reduce the risk just below TRG and was not intended to address all the risk.

This plan also includes the development of improved public education regarding the risk of living downstream of a dam. This will be developed and implemented in cooperation with local stakeholders and government bodies, as well as state and Federal emergency management agencies.

Mitigation requirements as determined the SFEIS will be part of this plan. In addition, the stilling weir will be stabilized and the approximate 66 anchors will be installed. The anchors are part of the FWAC condition, but will be implemented with CG funds because they are required to meet the estimated ECRA. The anchors will be designed and procured with the dam safety modification project to ensure compatibility. The remote operation of the crest gates will be implemented due to the risk reduction gained for life safety risk to dam personnel.

1.5.8. Risk Management Plan 8 - Stilling Basin with Super-cavitating Baffles & Higher Parapet Wall

This risk management plan consists of various features and risk management measures formulated to ensure stability of the stilling basin and the dam during extreme flood events. RMP 8 was formulated with the intention of achieving all essential USACE guidelines and is designed to safely pass the full IDF. This plan consist of every measure included in RMP 6 except for the remote operation of the spillway crest gates. This plan consists of a modified stilling basin with super-cavitating baffles; raised crest gates and bridge; higher (14-foot) parapet wall; and additional anchors in dam required for stability at the IDF. There are a few additional measures above and beyond RMP 6 that are included part of RMP 8 and are listed in Table 5-3.

This plan also includes the development of improved public education regarding the risk of living downstream of a dam. This would be developed and implemented in cooperation with local stakeholders and government bodies, as well as state and federal emergency management agencies.

Mitigation requirements as determined the SFEIS will be part of this plan.

In addition, this plan would include approximately 66 anchors which are part of the FWAC condition. The anchors will be implemented using CG funds because they are approved under the DSA project, but were never procured. The anchors are required to meet the estimated risk in the ECRA and FWAC. The anchors will be designed and procured with the dam safety modification project to ensure compatibility.

1.6. Evaluation and Comparison of RMPs

The evaluation criteria used to evaluate and compare the RMPs was based on the four Principles and Guidelines (P&G) criteria which includes effectiveness, efficiency, acceptability, and completeness. Two additional criteria, engineering (technical) feasibility and environmental effects were also added to further distinguish between plans.
Table 1-2: Comparison Summary of Risk Management Plans

<table>
<thead>
<tr>
<th>Plan</th>
<th>Pros</th>
<th>Cons</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Least Cost</td>
<td>Risk is unacceptable</td>
</tr>
<tr>
<td>2</td>
<td>Eliminates dam failure risk</td>
<td>Increased flood risk, environmental impacts and loss of significant authorized benefits</td>
</tr>
<tr>
<td>3</td>
<td>Reduces risk and meets essential guidelines</td>
<td>Less cost effective risk reduction than other plans, high cost, and environmental impacts</td>
</tr>
<tr>
<td>4</td>
<td>Reduces risk below TRG</td>
<td>Less cost effective risk reduction than other plans and environmental impacts</td>
</tr>
<tr>
<td>5</td>
<td>Reduces risk below TRG</td>
<td>Environmental impacts, more challenging to implement, and effectiveness has higher uncertainty</td>
</tr>
<tr>
<td>6</td>
<td>Most efficient plan to reduce risk below TRG with the least environmental potential impacts.</td>
<td>Performance uncertainty higher than some plans due to limiting modifications to existing footprint.</td>
</tr>
<tr>
<td>7</td>
<td>Low cost</td>
<td>Does not meet tolerable risk guidelines</td>
</tr>
<tr>
<td>8</td>
<td>Reduces risk and meets most all essential guidelines</td>
<td>High cost for similar risk reduction to RMP 4, 5, and 6.</td>
</tr>
</tbody>
</table>

1.7. Selected Plan
Risk Management Plan 6 (also referred to as Super Baffle plan in historical and technical documents), the modified stilling basin with the super-cavitating baffles, has been identified as the recommended plan (also referred to as the TSP) based on the efficiency criteria. RMP 6 is the most efficient plan at reducing incremental risk based on screening level cost estimates with the least amount of environmental risk and impacts. RMP 6 consists of structural measures working in unison to reduce risk and achieve the primary objectives of reducing the incremental risk of life loss. RMP 6 effectively reduces the incremental risk of life loss to below tolerable risk guidelines for Bluestone Dam. However, this plan is not designed to pass the IDF and therefore, does not meet all the essential USACE guidelines. A comparison of the final array of plans is shown in Table 1-2.

As previously noted, the purpose of the selected plan is to reduce the incremental risk associated with PFM #33, spillway monolith instability. The spillway design accomplishes incremental risk reduction by modifying the first stage stilling basin to prevent rock scour, reinforcing the second stage basin to minimize scour, and addressing scour concerns with overtopping of the training walls. These design features will be stabilized with approximately 1,750 bar and strand anchors, and uplift design loads will be reduced with approximately 375 total drains with approximately 80 of them located within the toe gallery. Specific details of RMP 6 design are as described in Chapter 6. In order to achieve the risk reduction presented in the with-project risk assessment, the additional anchors in the dam (approved in DSA project) must be installed. These additional anchors are part of the No Action (FWAC) Plan and not part of RMP 6.

The parapet wall was evaluated as an ALARP measure and was determined impracticable from an effectiveness standpoint. This evaluation is fully described in Section 5.2.6.4 (under the effectiveness evaluation) and is not part of RMP 6.

Based on risk informed decision making practices, it was determined the construction of the recommended plan will be implemented in two phases, Phase 5 and Phase 6 (if necessary). Phase 5 consists of the features described in RMP 6 including the construction of the under drains, but not
including the construction of the toe gallery. Based on the best available information and conservative assumptions, the anticipated performance of RMP 6 with the underdrain system only (i.e., no toe gallery) is estimated to achieve tolerable levels of risk. However, there is substantial uncertainty associated with this conclusion. The information needed to confirm these findings will be obtained only when a substantial portion of the new stilling basin is put in service and data regarding uplift pressures during pool loading scenarios is acquired. If adequate uplift relief is not achieved, Phase 6, construction of the toe gallery would be implemented. The Dam Senior Oversight Group (DSOG) supported the recommendation of this phased implementation approach. Additional details regarding design and implementation of the recommended plan can be found in Chapter 6.

![Rendered Aerial View of the Completed RMP 6](image)

**Figure 1-3: Rendered Aerial View of the Completed RMP 6**

**1.7.1. Policy Waivers Needed for Implementation of RMP 6**

**1.7.1.1. Hydraulics and Hydrology**

Engineer Regulation (ER) 1110-8-2 specifies that the selection of the IDF and the design of dam elements necessary to meet minimum safety requirements will conform to Standard 1 as defined in the referenced ER. Standard 1 applies to the design of dams capable of placing human life at risk or causing a catastrophe, should they fail. Standard 1 prescribes that the dam height with appropriate freeboard, spillways, regulating outlets, and structural designs will be such that the dam will safely pass an IDF computed from probable maximum precipitation (PMP) occurring over the watershed above the dam site.

The IDF for Bluestone Dam is the probable maximum flood (PMF). However, the TSP is designed to pass a lesser flood equivalent to 70% of the IDF. The TSP is justified because risks are judged to be tolerable. Risk for the TSP is below the tolerable risk guideline for life safety and risks are as low as reasonably
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practicable. This meets the intent of the inflow design flood policy in that the dam does not pose an unacceptable risk to life safety as defined by the tolerable risk guidelines in Engineer Regulation 1110-2-1156.

Formulating the TSP for a flood less than the IDF was decided on 3-4 March 2016 when the project delivery team (PDT) met with the Tier 3 Mega-Project Vertical Governance Senior Executive Board, which included key membership of the Dam Safety Senior Oversight Group (DSOG) to present the ECRA/FWAC. The meeting concluded that the spillway erosion potential failure mode was the only actionable dam safety issue. Overtopping risks were judged to be tolerable and therefore not actionable at this time. The decision is documented in the Memorandum for Record dated 8 February 2017 (Martin, PE, PMP, 2017). Further coordination with the MSC HH&C CoP representative and the HQ HH&C CoP Leader occurred during an ATR meeting at the Huntington District (LRH) office on 16 September 2016. Agreement was reached with the HH&C CoP that the proposed waiver was appropriate. The decision is documented in the Memorandum for Record dated 21 September 2016 (Koutsunis, 2016).

The request for waiver to the requirement to pass the IDF is considered part of this DSMS report and decision. Approval of the DSMS report constitutes approval of the policy waiver.

1.7.1.2. Structural
EM 1110-2-2100, Stability Analysis of Concrete Structures, 1 December 2005 requires approval from CECW-E to use new anchors to provide sliding stability to new concrete structures. As this study addresses modification of the existing dam (considered an existing structure), it is unclear whether a waiver request is required for anchoring of the new features of the recommended plan. Given the ambiguity in the intent of the guidance regarding what constitutes a "new" structure, a waiver request has been submitted for installation of the anchors for new concrete features that are part of the recommended plan (RMP 6). The new concrete features to receive new active (tensioned) anchors are the super-cavitating baffles, apron extension, and divider wall. This waiver request has been developed for the installation of the new anchors as outlined in Paragraphs 8-1 and 8-7b of EM 1110-2-2100. A memorandum requesting waiver for use of these anchors is being routed through LRD to HQUSACE for approval.

Also as noted in EM 1110-2-2100, the recommended plan is to be designed to the maximum design flood (MDF). However, for this study the MDF for the recommended plan is a lesser flood equivalent to 70% of the IDF as noted in the H&H policy waiver request. Therefore, an additional waiver is not needed for the structural design based on the 70% IDF.

1.7.1.3. Geotechnical
A waiver is being requested for the anchor design depth calculation based on EM 1110-1-2908 dated 30 November 1994. The memorandum serves as the formal request for CECW-CE approval of the alternative methods used to estimate anchor depths for the Bluestone Dam Safety Project. Using Formula 9-3 as proposed in Chapter 9 of EM 1110-1-2908 for design of anchors for the Bluestone Dam Safety Project produces embedment depths with constructability issues. As such, the memorandum dated 30 March 2017 request to use Formula 9-4 with the total of all anchor loads substituting for the single anchor load to calculate the anchor depth and then using 3-D MicroStation to confirm the design or 3-D MicroStation alone to determine anchor embedment depths. Both alternative methods rely on the weight of the rock mass to resist anchor forces as does Formula 9-3. (Morgan, P.E. & Martin, P.E., PMP, 2017)
1.7.2. Cost of Recommended Plan

1.7.2.1. Bluestone Total Project Cost (All Phases including DSA and DSMS)

The cost estimate for the project has been developed from detail in MCACES Second Generation (MII) cost estimating software. The estimate considers all prior and remaining project costs including engineering, design, and contract supervision & administration. The Northwest Walla Walla (NWW) Cost ATR Certification dated 22 June 2017 presents the Estimated Cost $561.9M at the FY17 price level, the Project First Cost of $897.5M at the FY18 price level, and the Total Project Cost is $1,017.4M fully funded. Within these amounts $323.3M has been spent through FY16, and another $45.15M obligated on existing contracts. See Table 1-3 for a cost summary by contract.

Table 1-3: Total Project Cost for Phases 1 through 6

<table>
<thead>
<tr>
<th>Contract</th>
<th>Estimated Cost FY2017 Price Level</th>
<th>Total Project Cost (Fully Funded)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prior</td>
<td>323.3 M</td>
<td>323.3 M</td>
</tr>
<tr>
<td>PRE Prior Expenditures</td>
<td>323.3 M</td>
<td>323.3 M</td>
</tr>
<tr>
<td>Remaining</td>
<td>561.9 M</td>
<td>694.1 M</td>
</tr>
<tr>
<td>Dam Safety Modification Study</td>
<td>1.9 M</td>
<td>1.9 M</td>
</tr>
<tr>
<td>Phase 3 Penstock Scour Pad</td>
<td>4.9 M</td>
<td>4.9 M</td>
</tr>
<tr>
<td>Phase 4 278 Multi-Strand Anchors</td>
<td>58.1 M</td>
<td>58.1 M</td>
</tr>
<tr>
<td>Phase 5 Stilling Basin</td>
<td>441.7 M</td>
<td>543.0 M</td>
</tr>
<tr>
<td>Phase 6 Toe Gallery</td>
<td>30.0 M</td>
<td>55.1 M</td>
</tr>
<tr>
<td>Miscellaneous Construction</td>
<td>5.2 M</td>
<td>5.8 M</td>
</tr>
<tr>
<td>Environmental Mitigation</td>
<td>9.6 M</td>
<td>10.7 M</td>
</tr>
<tr>
<td>Recreational Mitigation</td>
<td>5.0 M</td>
<td>5.5 M</td>
</tr>
<tr>
<td>Recreational Park</td>
<td>5.4 M</td>
<td>9.2 M</td>
</tr>
<tr>
<td>Grand Total</td>
<td>885.2 M</td>
<td>1,017.4 M</td>
</tr>
</tbody>
</table>

1.7.2.2. Bluestone DSMS Project Cost for RMP 6 (Phases 5 and 6)

A cost estimate was developed for the entire project phases 1 through 6, but this section only focuses on Phases 5 (RMP 6) and Phase 6 (toe gallery). The cost estimate for the recommended plan (RMP 6) was developed using MCACES Second Generation (MII) cost estimating software. The estimate considers all engineering, design, and contract supervision & administration for the DSMS recommendation that is included in this report. Table 1-4 shows the cost breakdown of activities for implementation of RMP 6.
Table 1-4: Cost Breakdown of Activities Required to Implement RMP 6

<table>
<thead>
<tr>
<th>Contract / Feature Account</th>
<th>Estimated Cost FY2017 Price Level</th>
<th>Total Project Cost (Fully Funded)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phase 5 Stilling Basin</td>
<td></td>
<td></td>
</tr>
<tr>
<td>04 - DAMS</td>
<td>441.7 M</td>
<td>543.0 M</td>
</tr>
<tr>
<td>30 - PLANNING, ENGINEERING &amp; DESIGN</td>
<td>92.7 M</td>
<td>111.6 M</td>
</tr>
<tr>
<td>31 - CONSTRUCTION MANAGEMENT</td>
<td>39.5 M</td>
<td>57.7 M</td>
</tr>
<tr>
<td>Phase 6 Toe Gallery</td>
<td></td>
<td></td>
</tr>
<tr>
<td>04 - DAMS</td>
<td>30.0 M</td>
<td>55.1 M</td>
</tr>
<tr>
<td>30 - PLANNING, ENGINEERING &amp; DESIGN</td>
<td>5.4 M</td>
<td>12.4 M</td>
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<tr>
<td>31 - CONSTRUCTION MANAGEMENT</td>
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<td>15.6 M</td>
</tr>
<tr>
<td>Miscellaneous Construction</td>
<td></td>
<td></td>
</tr>
<tr>
<td>04 - DAMS</td>
<td>5.2 M</td>
<td>5.8 M</td>
</tr>
<tr>
<td>30 - PLANNING, ENGINEERING &amp; DESIGN</td>
<td>1.8 M</td>
<td>2.1 M</td>
</tr>
<tr>
<td>31 - CONSTRUCTION MANAGEMENT</td>
<td>0.2 M</td>
<td>0.3 M</td>
</tr>
<tr>
<td>Environmental Mitigation</td>
<td></td>
<td></td>
</tr>
<tr>
<td>04 - DAMS</td>
<td>9.6 M</td>
<td>10.7 M</td>
</tr>
<tr>
<td>30 - PLANNING, ENGINEERING &amp; DESIGN</td>
<td>4.1 M</td>
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<td>31 - CONSTRUCTION MANAGEMENT</td>
<td>0.4 M</td>
<td>0.4 M</td>
</tr>
<tr>
<td>Recreational Mitigation</td>
<td></td>
<td></td>
</tr>
<tr>
<td>04 - DAMS</td>
<td>5.0 M</td>
<td>5.5 M</td>
</tr>
<tr>
<td>30 - PLANNING, ENGINEERING &amp; DESIGN</td>
<td>1.1 M</td>
<td>1.2 M</td>
</tr>
<tr>
<td>31 - CONSTRUCTION MANAGEMENT</td>
<td>0.3 M</td>
<td>0.3 M</td>
</tr>
<tr>
<td>Recreational Park</td>
<td></td>
<td></td>
</tr>
<tr>
<td>04 - DAMS</td>
<td>5.4 M</td>
<td>9.2 M</td>
</tr>
<tr>
<td>30 - PLANNING, ENGINEERING &amp; DESIGN</td>
<td>1.1 M</td>
<td>2.4 M</td>
</tr>
<tr>
<td>31 - CONSTRUCTION MANAGEMENT</td>
<td>0.3 M</td>
<td>0.7 M</td>
</tr>
<tr>
<td>Grand Total</td>
<td>497.0 M</td>
<td>629.2 M</td>
</tr>
</tbody>
</table>

The Total Project Schedule is organized by anticipated contract work packages to complete remaining project scope. Contract durations were developed either from real contractual periods of performance or by sequencing the major work activities. There are several key milestones of interest for the Bluestone Dam Safety Project. These are summarized in Table 2 and Figure 1 in the Total Project Cost Estimate Baseline (FY17) for the Supplemental Report Evaluation Report (Appendix I). The Dam Safety Risk Reduction Complete milestone defined as the minimal project features required to achieve reduction of risk. The current base project schedule estimates this Dam Safety Risk Reduction milestone competed late-FY31 (with 80% confidence completion by early FY34) for Phase 5. The baseline duration for Phase 5 is estimated to be ten years. Site conditions such as care and diversion of water requires non-concurrent sequencing of work which might be otherwise completed concurrently and more efficiently. Further, certain constraints have been assumed due to life risk considerations during construction. These
constraints also extend the contract duration. Performance of the stilling basin is assumed to be observed throughout construction and for a period of two years after completion of Phase 5 to determine whether Phase 6 will be needed. If Phase 6 is determined to be needed the Project Physically Complete is planned for 80% confidence completion by early FY-41.

1.7.3. RMP 6 – TSP Risk Assessment
As previously noted, after the ECRA and the FWAC risk assessment were revised based on the IDF update, a Vertical Team and Mega Project Tier 3 Governance meeting was held on 3-4 March 2016. This meeting also resulted in determining that only PFM 33 was actionable (Martin, PE, PMP, 2017). RMP 6 was selected as the TSP based on efficiency and therefore, the Cadre only performed a risk assessment on this plan. The risk reduction results for RMP 6 are shown in Figure 1-4. Additional risk reduction details for RMP 6 can be found in Chapter 7.

![Figure 1-4: RMP 6 Risk Plot Displaying APF and AALL for Incremental Risk](image-url)
2. BLUESTONE DAM BACKGROUND

2.1. Introduction

The Bluestone Dam Safety Modification Report (DSMR) has been prepared by the LRH and Great Lakes and Ohio River Division-Dam Safety Production Center (LRD-DSPC) with the assistance of the Louisville District (LRD) Risk Cadre and the Risk Management Center (RMC). This DSMR has been prepared in accordance with Engineering Regulation (ER) 1110-2-1156 (USACE Safety of Dams – Policy and Procedures and supplements the 1998 Dam Safety Assurance (DSA) Report.

A report was approved in 1998 to address a hydrologic deficiency as part of the Dam Safety Assurance (DSA) Program. Work approved in the 1998 DSA primarily consisted of raising the dam to prevent overtopping, increasing outflow capacity by utilizing the penstocks as an auxiliary spillway and stabilizing the dam with rock anchors and thrust blocks. Approved modifications began in 2000 and are currently scheduled to continue through the 2020’s. A brief summary of the DSA construction phases are listed below and additional details can be found in Table 2-2.

- Phase 1 of the phased modification project was awarded in 2000 and completed in 2004. Phase 1 included an access bridge over the stilling basin, mass concrete thrust blocks, extension of six penstocks and installation of three penstock gates.
- Phase 2A was awarded in 2004 and completed in 2007. Phase 2A consisted of a highway swing gate closure with parapet wall panels, an upgraded access road, a fishing pier for mitigation, a right abutment gravity wall, and utility line relocation.
- Phase 2B was awarded in 2005 and completed in 2011. Phase 2B installed 216 anchors and 3 remaining penstock gates.
- Phase 3 was awarded in 2010 and completed in 2017. This phase consisted of the installation of an auxiliary penstock stilling basin.
- Phase 4 was awarded in 2012 and includes installation of 278 high strength steel strand anchors in the spillway and non-overflow monoliths. Phase 4 is expected to be completed in 2019.

Since the DSA work was approved and awarded, USACE moved from a solely standards-based approach for its dam safety program to a risk informed dam safety approach. USACE developed a Dam Safety Action Classification (DSAC) system informed by the project’s incremental risk. Incremental risk is explained in detail in Chapter 4.

In order to determine Bluestone Dam’s DSAC, an Issue Evaluation Study (IES) completed in 2008 concluded that risks to the public were unacceptable even after implementation of the remaining DSA construction and further study was warranted. In 2013, a Baseline Condition Risk Assessment (BCRA) identified and confirmed an additional failure mode not addressed by the 1998 DSA study and Bluestone Dam became a DSAC 2 structure. Meaning, without intervention, progression toward probable failure could occur as a result of a flood event for an annual chance of exceedance (ACE) that is greater than 1 in 10,000 (1/10,000). The risk associated with the consequences from this scenario are very high.

This DSMR documents the decisions made during the Bluestone Dam Safety Modification Study (DSMS). The Bluestone DSMS further defines the Dam Safety issues, confirms Federal interest in operating the project, and recommends a risk management plan to reduce incremental dam safety risk to be within tolerable risk guideline, so the project can continue to operate as originally authorized. An array of risk management measures and plans were formulated, evaluated and compared to recommend a risk management plan (RMP) that reduces incremental risks to meet tolerable risk guidelines. USACE Tolerable Risk Guidelines are defined in Chapter 5 of ER 1110-2-1156 and are utilized to define what dam
safety issues present unacceptable risks. For the purposes of this report, failure is defined as a sudden, rapid, uncontrolled release of impounded water.

The remaining features approved by the 1998 DSA project, but not yet procured, include additional rock anchors (+/- 66) in the dam and a parapet wall across the top of the non-overflow sections. The additional rock anchors are further explained in Chapters 3 and 4 of this report and are considered as part of the FWAC condition. The parapet wall was reevaluated as part of the RMPs as an ALARP consideration and was determined impracticable from an effectiveness standpoint.

2.2. Report Organization

The remainder of Chapter 2 describes the purpose and need for the DSMS, provides a brief description of the authorized project purposes, and summarizes the location, description, history, current and future use of Bluestone Dam. A justification for continued Federal investment in Bluestone Dam is provided along with a recommendation to reduce incremental dam safety risk to meet tolerable risk guidelines.

The DSMS followed the six step framework of the civil works planning process contained in ER 1105-2-100 (Planning Guidance Notebook) as adapted for addressing Dam Safety issues in ER 1110-2-1156. The remainder of the draft DSMR is organized according to these steps.

- Chapter 3 provides information on the issues and opportunities, as well as goals, objectives and considerations of the DSMS. This chapter presents the purpose and need for Federal action. This chapter also defines the existing and future without actions condition utilized to develop the existing and future without Federal action condition (FWAC) risk assessment presented in Chapter 4.
- Chapter 4 describes the ECRA and FWAC risk. This chapter also includes information on what the existing risks are and what incremental risks are expected if no action (FWAC) was taken by the USACE Dam Safety program. In this chapter the significant Potential Failure Modes (PFMs) requiring action are outlined. Significant PFMs are those ways the project could fail which will be combined with the potential consequences of failure to determine risk.
- Chapter 5 describes the formulation of RMPs, the evaluation and comparison of RMPs, and the rationale for the tentatively selected plan (TSP).
- Chapter 6 describes the design features of the selected plan (TSP). This chapter presents the design, cost, implementation, operation, environmental resources, real estate, and cost-sharing of the TSP along with additional future actions to address non-breach risk.
- Chapter 7 estimates the dam safety risk assessment of the selected RMP.
- Chapter 8 summarizes the reviews conducted as part of the DSMS.
- Chapter 9 is a list of references.
- Chapter 10 is a list of acronyms used in the report.

2.3. Dam Safety Action Classification

All USACE dams are managed in a dam safety portfolio risk management process. As part of the overall USACE portfolio a Dam Safety Action Classification (DSAC) system is used as a systematic approach to evaluate and assign action classification. There are five action classifications ranging from those critically near failure (DSAC 1) to those considered to have very low risk associated with dam failure (DSAC 5).

Bluestone Dam is currently identified as a DSAC 2 (high urgency), due to the fact that progression toward probable failure could be initiated by a flood event with an ACE that is greater than 1 in 10,000 (1/10,000) and the consequences of failure are high. A DSAC 2 can also be identified when the incremental risk (combination of life or economic consequences with likelihood of failure) is high, which is the case for Bluestone Dam.
In 2008, Bluestone was assigned a DSAC 2 and then presented the Baseline Condition Risk Assessment (BCRA) to the Dam Senior Oversight Group (DSOG) in July 2013 where the classification was reaffirmed. After the DSOG meeting, LRH received a memorandum in September 2013 from the HQ Dam Safety Officer stating that “the structure should remain classified as a DSAC 2” (Dalton, P.E., SES, 2013). As part of the recommendations of this memo, the team updated the hydrologic hazard and the results led the team to reevaluate the risk associated with the dam. In March 2016 the risk assessments (ECRA and FWAC) were revised and did not support a need to present to the DSOG because it concluded the same risk characterization. The Tier 3 vertical team agreed the structure was to remain a DSAC 2 in March 2016 and gave verbal direction to continue the DSMS with the ECRA and FWAC results (Martin, PE, PMP, 2017). Given this classification, actions to reduce risks to public safety to achieve the USACE tolerable risk levels and to reduce (minimize) the threat to economic, societal and environmental consequences in the region need to be considered. The purpose of this study is to investigate and identify the appropriate actions to manage dam safety risk. This report documents the rationale for the proposed actions as defined by Engineering Regulation (ER) 1110-2-1156.

2.4. Project Location and Study Area
Bluestone Lake, which is created by Bluestone Dam, is a flood risk management reservoir on the New River near City of Hinton, Summers County, West Virginia. Bluestone Dam, is a conventional concrete gravity dam. It is within the New River Basin, which is a sub-basin of the Kanawha River Basin. The dam is located approximately one and a half miles upstream of the City of Hinton and a half mile upstream of the confluence of the New and Greenbrier Rivers. The project began operation in 1949.

Bluestone Dam helps control a 4,600 square mile drainage basin. The dam helps to reduce flood risks along the New, Kanawha and Ohio River Basins. These are the largest interior river valleys in the state of West Virginia and encompass the state capitol of Charleston, major manufacturing and chemical industries, and approximately 165,000 population at risk (PAR). Figure 2-1 provides the map of Bluestone Dam’s geographic location, the watershed, rivers, other basin reservoirs and key cities and towns. The study area for the DSMS includes the New and Kanawha River valleys from Narrows, Virginia to Point Pleasant, West Virginia and the Ohio River valley to Greenup, Kentucky.

The dam is intended to store flood waters up to elevation (EL) 1,520-ft NGVD 29. Upon reaching this elevation, the crest gates will fully open to maximize outflows. Although the dam is in place, there will still be non-breach flooding downstream.

1 All elevations in this report are presented in NGVD 29 datum unless stated otherwise. Justification for utilization of this legacy datum and conversion to the most current datum can be found in a memorandum from CELRH-EC-TG dated 22 February 2017 (Miller & Sullivan, 2017). The relationship between the legacy datum (NGVD 29) and NAVD 88 is 0.567 feet with the legacy datum being the higher elevation (Caldwell, 2017).
2.5. Authorized Project Purposes

The Bluestone Reservoir Project was authorized by the Flood Control Acts of June 22, 1936, and June 28, 1938, approving the provisions of Executive Order No. 7183-A, dated September 12, 1935. This act directed the construction of the reservoir as a flood control and power project in accordance with the general plans presented in the supplemental report on the Kanawha River published in House Document No. 91, Seventy-fourth Congress, first session. A project report on the Bluestone Reservoir was submitted December 26, 1936, and approved May 13, 1937. The Government has the responsibility to furnish lands, easements, rights-of-way, relocation and disposal areas (LERRD) pursuant to Sections 2 and 4 of the Flood Control Act of 1938, Stat. 1215. The cost sharing provisions of Section 1203 of the Water Resources and Development Act of 1986 (P.L. 99-662) do not apply to this project.
Bluestone Dam – Final DSMR

Bluestone Reservoir is a part of a comprehensive reservoir system for flood control on the mainstem of the Ohio River and its tributaries. It was implemented through multiple authorizations for flood control, low flow augmentation, hydropower development, recreation, and fish and wildlife conservation. After the construction of Bluestone Dam, additional laws, regulations, and authorizations were added that are now considered part of the authorized project. The following list contains legislative authorities for the authorized purposes. Details of the authorized language can be found in Appendix N.

**Flood Control, Low Flow Augmentation, Power Development and Drift and Debris**
- Franklin D. Roosevelt’s Executive Order (E.O.) 7183A – September 1935
- Flood Control Act of 1936 (P.L. 74-738)
- Flood Control Act of 1938 (P.L. 75-761)
- Water Resource Development Act (WRDA) of 1992 (P.L. 102-508), Section 102(ff)
- Section 357 of WRDA 1996 (P.L. 104-303)
- Section 547(a)of the WRDA of 2000, (P.L. 106-541)

**Construction Operation and Maintenance of Public Park and Recreation Facilities, Recreation Enhancement**
- Flood Control Act of 1944, Section 4 (P.L. 78-534)
- WRDA 1988, Section 6 and 47 (P.L. 100-676)

**Fish and Wildlife Conservation**
- Fish and Wildlife Coordination Act of 1958 (P.L. 85-624)

2.6. Current Use of the Project and Projected Future Use
The dam serves as one of the primary means of providing flood risk management benefits to hundreds of thousands of people downstream of the dam. Abundant recreation opportunities are provided to the public as a result of the reservoir and recreational facilities both upstream and downstream of the dam. The downstream environmental resources benefit from the low flow augmentation due to the minimum flow requirements. The drift and debris tower facilitates acceptable management of drift and debris from the reservoir to pass downstream. The project was designed to add a hydroelectric generating plant. Section 547(a) of the WRDA of 2000 (P.L. 106-541) authorized the installation of a non-federal hydroelectric generating plant. A request to modify the project to install this plant has not been requested pursuant the 33 USC 408 and its implementing procedures outlined in Engineering Circular 1165-2-216.

2.7. Warranted Continued Federal Investment
Bluestone dam and reservoir provides benefits in all the categories of the authorized project purposes, with the exception of hydropower that has not been developed to date. Those benefits to date support warranted continued Federal investment to maintain the existence of the dam and continued operation and maintenance. The remainder of this section summarizes the historical and expected benefits. Because the dam will be modified in the existing footprint and continue to provide the benefits as originally authorized, modifications to the existing project purposes (authorizations) are not warranted at this time.

2.7.1. Flood Risk Management Benefits, Population at Risk and Potential Economic Damages
Flood risk management benefits (damages prevented) accruing from when the project began operation through fiscal year 2015 are estimated at over $2 billion in real dollars (over $5 billion indexed to FY15
dollars). Those historic damages prevented from 1949 to 2015 were brought to FY 2015 price level and averaged to yield an annual benefit of $87 Million. It is expected the dam will continue to provide a similar amount of annual flood risk management benefits. The flood risk management benefits are realized in the communities of Hinton, Gauley Bridge, Montgomery, Kanawha City, Charleston, South Charleston, St. Albans, Nitro, Winfield, Point Pleasant, and many communities along the Ohio River downstream from its confluence with the Kanawha River. Many within these communities have limited ability to recover from flood events. These communities have a higher social vulnerability than other communities. Due to the flood risk management benefits the dam still provides today (protection of lives and property) continuation of existing project purposes are warranted.

Additional justification to modify the project is provided from the existing condition risk assessment (ECRA). The ECRA concluded that the dam safety risk in its current state is above the USACE tolerance threshold and the dam poses a significant risk to public safety. When considering approximately 165,000 people are at risk given a breach at maximum pool levels and the associated direct economic damages of over $19 Billion resulting from a failure, modifications to the dam are warranted.

2.7.2. Recreation Benefits
The project is also authorized for recreation that provides benefits for visitors pursuing water related activities including fishing, hunting, boating, water skiing, and picnicking. The project has a permanent pool with an average surface area of 2,040 acres, 21,954 acres of land, and 28 miles of shoreline. Using annual project visitation data obtained from the Corps’ Operation and Maintenance Business Information Link (OMBIL) the average annual visitation during the period from 2008 to 2012 was approximately 1.7 million visits. Applying the Unit Day Value methodology (EGM15-03), the benefit annually from recreation visitation is estimated to be $16.5 Million. Similar recreation benefits are expected in the future.

2.7.3. Fish and Wildlife Conservation
This project is authorized for fish and wildlife conservation. In broad terms, the goals under the Fish and Wildlife Coordination Act of 1958 are intended to promote the long-term wellbeing of populations of the plant and animal species native to the project area and the maximum sustained enjoyment of these populations by the public. The Bluestone Dam is located on the New River which contains a high diversity of excellent quality fish and wildlife habitats. The dam supports and provides benefits to the aquatic habitats below the dam which are of high quality and considered by the U.S. Fish and Wildlife Service as unique and irreplaceable. The New River downstream of the dam (66 miles to its confluence with the Gauley River) supports one of the best warm water fisheries in West Virginia. The smallmouth bass within this reach of the New River supports one of the best fisheries for this species in the United States (USFWS, 1998 and 2017). Sustaining flows or maintaining minimum flow conditions, drought contingency plans and other strategies are expected to provide these benefits into the future.

2.7.4. Water Supply Benefits
The Bluestone Reservoir does not have storage space specifically allocated to water supply; however per the Reconnaissance Report for Water Supply Storage, Bluestone Lake, West Virginia & Virginia, dated April 1993, a water supply intake was approved for the greater Princeton, West Virginia and surrounding areas not to exceed 15 MGD. The water supply intake located near the mouth of the Bluestone River at EL 1,401-ft and provides potable water to approximately 12,000 to 13,000 customers (households/businesses) and pumps an average of 2.5 to 3 million (maximum capacity of pumps is only 5 million) gallons per day. While not an authorized project purpose, the water supplied from Bluestone Reservoir does provide potable water to these customers and is expected as a continued benefit.
2.7.5. **Hydropower Benefits**

Hydropower is an authorized project purpose, but has not been developed to date. However, it would provide benefits into the future if implemented.

2.7.6. **Summary Authorized Project Purposes Warrant Continued Federal Investment**

Based upon these benefits, it is reasonable to conclude the project has historically served to provide the authorized purposes with the exception of hydropower production which has not been developed to date. It is important to note that the project is continuing to provide the authorized purpose of flood risk management; however, currently it does not function to the full level of design because of dam safety issues. The change in operation has been addressed by water control plan (WCM) deviation which is in place until Phase 4 of the DSA construction is completed. With the completion of Phase 3 construction, this deviation is currently being updated to reevaluate the pool level restriction, the outflow restriction and to include operation of the penstocks as an auxiliary spillway. Upon completion of Phase 4 construction, the full design flood storage elevation will be restored but an update to the WCM will be required to account for the reduced safe discharge capacity of the primary spillway and to add penstocks as an auxiliary spillway. For details of the timeline for WCP activities see Table 6-3 and Figure 6-15.

Given the substantial benefits provided by the dam to include flood risk management benefits (protection of human life and property), recreation, fish and wildlife conservation, drift and debris management, water supply; potential hydropower along with the investment made toward construction of remedial dam safety measures (DSA efforts), continued Federal investment to address the remaining dam safety issues is warranted.

2.8. **Bluestone Dam Description**

The project began operation in December 1949 and controls a 4,600 square mile drainage area. The dam is a straight, concrete gravity structure with a maximum height of 165 feet above the stream channel (See Figure 2-2). The length of the dam is 2,048 feet at EL 1,535-ft and is made up of 55 independent monoliths comprising of right and left abutments, a non-overflow section, and a penstock section originally intended for hydropower (recently modified to become an auxiliary spillway), an assembly bay, and a primary spillway.

As originally envisioned, Bluestone Dam, was to hold a permanent normal pool to EL 1,490-ft for hydropower generation with 30 feet of flood storage capacity (approximately 1 inch of runoff over the 4600 square miles above the project) up to EL 1,520-ft. However, because hydropower was not implemented, this normal pool was reduced to EL 1,410-ft in the summer and EL 1,406-ft in the winter, giving the dam between 110-114 feet of flood storage capacity (approximately 2.43 and 2.47 inches of runoff over the 4,600 square miles above the project) and it has operated as such since (See Figure 2-3). Between, the months of April and November, summer pool is maintained at EL 1410-ft to facilitate recreation on the lake when the project is not storing or releasing flood waters. Between the months of December and March, the pool is lowered to a winter pool at EL 1,406-ft to increase flood storage. During this scheduled drawdown, the surface area available for recreation on the lake is reduced. Within this operation, outflows are restricted to a minimum flow of

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Figure 2-2: Aerial of Bluestone Dam
610 cubic foot per second (cfs), a maximum without directive (from the District’s Water Management Team) being 45,000 cfs, and a maximum with directive of 90,000 cfs.

The project’s discharge capacity is accomplished through 16-double gated sluices, gated primary spillway and a six-gated penstocks of the auxiliary spillway. The six, 18-ft diameter penstocks were installed as part of the original construction for hydropower generation but this was never implemented. Bulkheads were originally installed near the intake side but these have been removed and replaced with gates at the downstream outlet. These gates are hinged at the bottom and are intended to be opened under reservoir load but cannot be closed until the reservoir recedes to a level at or below the intake invert. This work was completed in order to supplement discharge as an auxiliary spillway. The discharge capacity of the penstocks is 150,000 cfs with the reservoir at the top of dam (EL 1,535-ft). Normal operation of the reservoir is achieved through 16 double-gated sluices with a maximum total discharge capacity of 72,000 cfs (at pool EL 1,517-ft with no spillway crest gate flow). See Table 2-1 for detailed pertinent data about the dam. The original design discharge capacity of the dam is 430,000 cubic feet per second (cfs) which is the same as the original estimated peak inflow.

Figure 2-3: Reservoir Diagram (All Elevations are in NGVD 29)

The primary spillway section is 790-ft wide with a crest elevation of EL 1,490-ft. and consists of a two stage basin. The first stage basin includes a concrete apron with two rows of baffles and an end-sill, an unlined 182-ft stilling pool maintained by a 23 feet high stilling weir and two training walls. The second stage basin
is comprised of a concrete apron with two rows of baffles and an end-sill. See Figure 2-4 for a graphical depiction of the dam’s primary features.

The auxiliary penstock spillway, has six extended, modified and gated penstocks for emergency discharge up to 150,000 cfs. Discharge from the auxiliary penstock spillway is guided (trained) through the divider walls, over the scour protection in the stilling basin, across two rows of baffles before flowing over an exit pad to the river.

The founding elevation of the dam is approximately EL 1,358-ft where it rests on the Stoney Gap Sandstone, the basal member of the Hinton Formation. This sandstone is very hard, gray, dense, well-cemented siliceous quartzitic sandstone (also called orthoquartzite) with occasional thin interbeds and lenses of black shale (interbedded orthoquartzite and shale).

Regional topography around the lake ranges from EL 2,600-ft to EL 1,300-ft near the dam. In the event of a dam breach, inundation occurs downstream of the dam and flows through a valley and gorge type landscape impacting downstream communities and agricultural lands. For a more thorough description of Bluestone Dam’s existing features, refer to Section 3.2.2.1.
### Table 2-1: Bluestone Dam Pertinent Data (Existing Condition)

<table>
<thead>
<tr>
<th>Bluestone Dam</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Dam Type</strong></td>
<td>Concrete Gravity (with 55 monoliths)</td>
</tr>
<tr>
<td><strong>Maximum Height</strong></td>
<td>165 feet</td>
</tr>
<tr>
<td><strong>Length</strong></td>
<td>2,048 feet</td>
</tr>
<tr>
<td><strong>Top of Dam Elevation</strong></td>
<td>1,535.00 feet NGVD 29 (1,534.45 feet NAVD 88)</td>
</tr>
<tr>
<td><strong>Primary Spillway Crest Elevation</strong></td>
<td>1,490-ft NGVD 29 (1,489.45-ft NAVD 88)</td>
</tr>
<tr>
<td><strong>Primary Spillway Width</strong></td>
<td>790-ft total (630 feet of opening)</td>
</tr>
</tbody>
</table>
| **Primary Spillway Type** | Gated concrete ogee spillway in channel section of dam. 21 vertical lift gates (30’ x 31’)
Invert Elevation – 1,490-ft NGVD 29 (1,489.45-ft NAVD 88) |
| **Auxiliary (Penstocks) Spillway** | Gated penstocks through right inake section of dam. Six, 18’ diameter penstocks with bottom hinged gates at outlet (19’-6” x 14’)
Invert Elevation – 1,410-ft NGVD 29 (1,409.45-ft NAVD 88) |
| **Outlet Works**       | Gated sluices through spillway section of dam. 16 Double Gated Sluices (5’-8”x10’)
Invert Elevation – 1,389-ft NGVD 29 (1,388.45-ft NAVD 88) |
| **Hydrology**          |  |
| **Drainage Area**      | 4,600 Sq. Mi.        |
| **Max. Historic Release** | 62,070 cubic feet per second (cfs) |
| **Max. Historic Pool Elevation** | 1,506.04-ft NGVD 29 (1,505.49-ft NAVD 88) on 6 April 1960 |
| **Reservoir Levels**   |  |
| **Pool Descriptions**  | Elevation (ft) | Storage(acre-ft) |
| **Winter Pool**        | 1,406 NGVD 29        | 16,624         |
| **Summer Pool**        | 1,410 NGVD 29        | 23,049         |
| **Pool of Record**     | 1,506 NGVD 29        | 494,100        |
| **Top of Active Storage Pool (Flood Control Pool)** | 1,520 NGVD 29 (1,519.45 NAVD 88) | 614,000 |
| **Spillway Design Flood** | 1,523 NGVD 29 (1,522.45 NAVD 88) | 641,900 |
| **Top of Dam**         | 1,535 NGVD 29 (1,534.45 NAVD 88) | ~789,000 |
| **IDF/PMF**            | 1,553 NGVD 29 (1,552.55 NAVD 88) | N/A |

1A 1 foot tall curb raises the effective top of dam for overtopping to EL 1,536-ft. (2013 BCRA, Section 6.2.2.)
2.8.1. Construction History
Bluestone Dam began construction in 1942 and was completed in 1948. There was an approximate two year period of time (March 1, 1944 to January 2, 1946) where dam construction was suspended due to higher Federal priorities (World War II). Operation of Bluestone Dam began in July 1949, and minimum pool was attained in August 1949. The crest gates were installed in 1950. The original construction project was 100% federally funded in the amount of $29.5 million (1949 Price Level).

2.8.2. Performance History
Only two flood events have resulted in impoundments on the crest gates. The first event is the pool of record (EL 1,506-ft) that occurred in April 1960. Dam tender records show that numerous crest gate operations were made to help pass drift and debris. These operations were initiated when the pool receded to EL 1,497-ft. A maximum of two gates were fully opened during this period, and opening of adjacent gates was avoided. Discharge data shows that the maximum outflow was approximately 62,000 cfs. In 1996, after the peak pool (EL 1,494-ft) had been observed, individual crest gates were opened to help pass drift and debris. It should be noted that this peak pool elevation was a bit higher because additional water was deliberately stored so that passing debris through the crest gates could be attempted and evaluated. Total outflows from this event never exceeded 60,000 cfs. The project has operated with all sluice gates fully opened regularly. Indications of instability or signs of distress such as differential movement at monolith joints have never been observed. However, no such measurements were taken during flood events to discern whether temporary deflections occurred during loading.

It should be noted that, although this dam has performed satisfactorily during high pools (pool of record of EL 1,506-ft), past performance may not be indicative of future response to the same pool loading. Anchor installation as part of DSA construction has filled many rock fractures in the foundation which has likely altered the distribution of uplift pressures beneath the dam. Even if structural modifications were not done, seepage through rock fractures usually causes calcification in the drains that reduce their efficiency and increases uplift beneath the dam.

Leakage through lift joints in Monoliths 27 and 29 has been observed when the pool reaches EL 1,470-ft. A stability analysis was performed in 1979 on these monoliths with the assumption that a cold joint was present at this elevation and connects to an existing vertical crack observed atop the four foot diameter shafts above gate cylinders in the operating chamber (EL 1,438-ft). The conclusion made at the time was the monoliths were adequate for these conditions and no remedial action or further analysis was recommended. More recent analysis of internal stability using current criteria also appears to show that there are no internal stability issues.
2.8.3. Routine Operation and Maintenance

Dewatering of the stilling basin has occurred three times (1979, 1994, and 2004) as part of routine periodic inspections (PI). Historical photos show intermittent accumulation of rock and debris just downstream of the stilling basin apron end sill in 1979 and 1994. During the 2004 dewatering, additional rock and debris was noted around the stilling basin apron end sill. Note that crest gate flow occurred between the 1994 and 2004 dewatering.

Foundation drainage is maintained by periodic reaming or pressure washing of drains in the inspection gallery. Most recently, drains were reamed (overdrilled by approximately ¼-inch) in 2009, which increased drainage from the gallery from approximately 3 to 6 gpm. Since then, 12 drains and one uplift cell have been lost due to intersection by an anchor borehole or infiltration by grout during Phase 4 construction which, when combined with losses from Phase 2B, result in some monoliths having as little as 50% of the original number of drains. Currently there are 284 functioning foundation drains in the inspection gallery, although anchor installations are still on-going and additional drain losses are expected to continue. Reaming of all remaining open drains is planned to be performed approximately every 10 years, due again in 2019. Because of the close spacing between drains and adjacent anchors, drilling of new drains is not considered to be feasible. Additional details are provided in Appendix E, paragraph 5.1.

All spillway crest gates are operated on a yearly basis, but have never been tested under full hydrostatic load. The spillway gates successfully operated with two to four feet of water on them in 1996. Periodic Inspection (PI) Reports (2009 & 2014) reported areas of corrosion and flat spots from wear on the wire ropes. As a result, the wire ropes on the 21 crest gates were replaced in 2015. It was also concluded from PIs that the intermediate gears and pinions have a lot of wear, radial grooves, and pitting. The brake on crest gate 9 had difficulty stopping the gate and was found to have a heavy grooved brake wheel and worn brake pads and the pads were replaced in 2013. Two worm gear reducers have been rehabilitated since 2014. The most recent PI in 2015 noted the crest gates were in good condition with only minor debris build up and corrosion with only touch up painting recommended.

The penstock gate latch release mechanisms are exercised once a year but the gates are never opened as they cannot be reclosed until the pool drops below the upstream penstock intake invert of EL 1,410-ft and requires the use of a crane or other equipment to swing the gate back up into the closed position. So during the yearly exercise the gates are dogged off using chain falls prior to operating the latch release. There are multiple means in place to prevent accidental release.

The drift and debris gates are sluice/slide type gates with hydraulic cylinder actuators and are kept in the closed position and are operated approximately four times a year. This arrangement means that the cylinders are in the extended position with rods exposed most of the year. These cylinders have ceramic coated rods of a type that has history of failures due to corrosion occurring under the ceramic coating on rods that are exposed for long periods of time and are not regularly treated with a sealant. The cylinders are inspected by boat or barge from the upstream side, which only gives good access to the lower cylinders. The cylinder can be inspected by man basket lowered from the derrick crane but such an inspection has not occurred. The cylinders that have been closely inspected have been reported to be in good condition.

The original penstocks were not inspected for corrosion or section loss. In the localized areas where the upstream bulkhead was removed and where the new extension was added to the downstream end, the existing coal tar paint system (believed to be lead based) was removed and painted with vinyl paint. The
penstock plate thickness of the new extension is 3/4". According to the work-as-constructed drawings, the plate thickness of the existing penstock varies from 5/8" thick to 15/16" thick.

2.9. Major Modifications

2.9.1. Dam Safety Assurance (DSA) Project
The performance of Bluestone Dam was evaluated in the mid 1990’s under the Bluestone Dam Safety Assurance (DSA) Study. Since the dam was originally designed and built, advances in hydrology, rock mechanics and stability analyses of concrete gravity structures concluded the dam was hydrologically deficient and had stability issues. The DSA study recommended a plan to address these issues and was approved for an estimated total project cost of $91 million (October 1996 Price Level). The major features of the DSA approved plan consisted of the following:

- High-capacity, multi-strand anchors;
- Mass concrete thrust blocks against the downstream face of the dam;
- Convert existing six penstocks into an auxiliary spillway;
- Additional gravity monolith on the east abutment; and
- Raise the non-overflow portions of the dam by eight feet.

The DSA project was divided into five phases. The five phases are listed in Table 2-2 along with the approximate cost and construction period. Also, prior to the initiation of Phase 2B, a test anchor program was conducted where four stabilizing production anchors were installed in Monolith 46 of the dam.

Table 2-2: Summary of Bluestone Dam Safety Assurance Construction Phases and Activities

<table>
<thead>
<tr>
<th>DSA Project</th>
<th>Construction Activities</th>
<th>Construction Contract Cost*</th>
<th>Construction Period</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phase 1</td>
<td>Construction of mass concrete thrust blocks downstream side of monoliths 15-21, extension of six penstocks, and three of the six penstock gates were installed.</td>
<td>~ $20M</td>
<td>Sep 2000 – Nov 2004</td>
</tr>
<tr>
<td>Phase 2A</td>
<td>Installation of a swing gate closure and two associated parapet wall panels across State Route 20 on the left abutment, upgraded the access roadway to the stilling basin, installation of a new fishing pier, construction of a gravity wall (right abutment), relocation of primary power and telephone lines.</td>
<td>~ $7.5M</td>
<td>May 2004 - Dec 2007</td>
</tr>
<tr>
<td>Phase 2B</td>
<td>Installation of 216 high capacity anchors in critical monoliths, installation of gates on the three remaining penstocks, cleaning of existing drains, and installation of 32 gallery drains in the spillway area along with 16 uplift cells.</td>
<td>~ $60M</td>
<td>May 2005 - Nov 2011</td>
</tr>
<tr>
<td>Phase 3</td>
<td>Installation of a scour pad stilling basin downstream of the penstock extension, construction of two training walls adjacent to each side of the scour pad, and addition of five divider walls and two partial divider walls (designed to separate flow from penstock discharge), and the incorporation of a transition section and baffle blocks with an end sill into the scour pad.</td>
<td>~ $75M</td>
<td>Sep 2010 - Feb 2017</td>
</tr>
<tr>
<td>Phase 4</td>
<td>Installation of approximately 278 high capacity steel strand anchors in the spillway and non-overflow monoliths.</td>
<td>~ $95M</td>
<td>Under Construction (Sep 2012 - Expected completion Oct 2019)</td>
</tr>
</tbody>
</table>

*These are real (unindexed) contract cost to include any issued modifications and/or options.
Figure 2-6: Bluestone Dam Safety Assurance (DSA) and Dam Safety Modification Study (DSMS) Components
2.9.2. Drift and Debris Tower

When construction restarted in 1946 the operational plan for the project was revised to include the capability of handling drift and debris through the sluice gates. Since hydropower was not implemented, the reservoir pool was reduced by 80 feet to take advantage of the flood risk management benefits. This rendered the original trash chute non-functional. At the time, no physical adjustments had been made to the project to maintain normal passage of drift and debris during high flow periods. Drift and debris was released through the sluice gates with the revised pool elevation. During flood events the drift and debris would rise to the higher pool elevations causing it not to pass through the sluices; therefore, it accumulated when flood storage was used. After the pool receded, project personnel had to manually assist the drift and debris to pass through the sluice gates once the pool returned between EL 1,406-ft and EL 1,410-ft. This resulted in large quantities of drift and debris passing downstream when the river stage was normal and there was inadequate velocity to carry the material without causing snags downstream. Drift and debris was passed through the sluices at lower flow conditions resulting in the accumulation of snags downstream. This was particularly an issue in the Hinton-Sandstone Falls reach.

In December 1996, a Bluestone Drift and Debris Evaluation Study was completed to further evaluate this issue. As a result of the study, a multi-level intake tower was approved for construction. A 10-foot tunnel through the dam was also approved to pass drift and debris during higher pools to simulate natural conditions. The drift and debris contract was awarded in 2001 and completed in 2005.

The construction activities included installation of an intake tower with nine multi-level hydraulically operated gates located at monolith 35 on the upstream side of the dam. There is a 10-foot tunnel installed through the dam to allow drift and debris to pass from the lake to the New River. The gates can either be operated from on top of the dam or from Operation’s boat on the lake through hand held controls. The hydraulic power unit (HPU) is located in Pylon 2. The cost to construct the tower and tunnel was approximately $9M.

2.9.3. Additional Dam Safety Maintenance Items

A detailed list of maintenance items and remedial measures is maintained and included in each PI report in an appendix titled “History of Remedial Measures.” Below is a list of major maintenance items that have been conducted at the project since it began operation in 1949. Several of these items (cleaning of foundation drains, rehab of uplift cells, and instrumentation enhancements) were initiated as a result of the dam safety assurance (DSA) study to address dam safety concerns. Items listed that do not include costs were minimal or details were not available.

- Dewatering and inspection of upper stilling basin (1979, 1994 and 2004)
- Installation of new and rehab of uplift cells (1999 and 2004)
- Remote instrumentation installation (2000) - $105K
- Painting of crest gates and other items (1997) - $2M
- Replacement of Crest Gate Controllers (1984) – $51K
- Rehab of roller chains for crest gates (2005)
- Replacement of derrick crane (2012) - $1.4M
- Replacement of motor control center (2014) -$180K
- Rehabilitation of crest gate right angle gear reducers on gates 7 and 9 (2014)
- Wire rope replacement on 21 crest gates (2015) - $630K
- Drained, flushed, and refilled oil in all crest gate right angle gear reducers (2015)-$6K
2.10. Interim Risk Reduction Measures (IRRM)

The Interim Risk Reduction Measure (IRRM) Plan for Bluestone Dam was completed in 2014 in accordance with ER 1110-2-1156 (US Army Corps of Engineers, Interim Risk Reduction Measures Plan, 2014).

The Interim Risk Reduction Measures listed in Table 2-3 were developed by the PDT and are intended to be feasible measures that have been or can be implemented in a timely fashion in order to reduce the exposure to risk prior to implementation of remaining DSA or DSMS features. Each measure includes a discussion of triggers for action, status, reduction in risks, impact on project purposes, environmental and economic impacts and cost. The details and status of the IRRMs can be found in the IRRM Plan (US Army Corps of Engineers, Interim Risk Reduction Measures Plan, 2014).

Table 2-3: List of IRRMs, Costs and Status

<table>
<thead>
<tr>
<th>Interim Risk Reduction Measure</th>
<th>Cost ($1,000)</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Update Water Control Manual (pool and discharge restrictions)</td>
<td>TBD</td>
<td>On Going Reevaluation</td>
</tr>
<tr>
<td>Emergency Exercise</td>
<td>$123 and $400</td>
<td>Completed</td>
</tr>
<tr>
<td>Emergency Action Plan</td>
<td>$2</td>
<td>Completed</td>
</tr>
<tr>
<td>Develop Communication Plan</td>
<td>$2.5</td>
<td>Completed</td>
</tr>
<tr>
<td>Update Inundation Maps</td>
<td>$150</td>
<td>Completed</td>
</tr>
<tr>
<td>Instrumentation Observation Schedule and Surveillance Plan</td>
<td>$2</td>
<td>Completed</td>
</tr>
<tr>
<td>Labor Resource Plan</td>
<td>$5</td>
<td>Completed</td>
</tr>
<tr>
<td>Extend Apron Foundation Drains</td>
<td>--</td>
<td>Abandoned</td>
</tr>
<tr>
<td>Replace all Wire Ropes for Spillway Crest Gates</td>
<td>$630</td>
<td>Completed</td>
</tr>
<tr>
<td>Gate Reliability – Crest Gate Machinery Rehabilitation</td>
<td>$1,200</td>
<td>Awaiting Funding</td>
</tr>
<tr>
<td>Provide Fall Protection at Crest Gate Dogging Devices</td>
<td>$12</td>
<td>Awaiting Funding</td>
</tr>
</tbody>
</table>
3. PROBLEMS, OPPORTUNITIES AND PLANNING ASSUMPTIONS

This chapter provides general social, economic and environmental characteristic of the study area and describes the assumptions that were used to identify the hydrologic loading on the dam. Problems, opportunities, objectives, and constraints are included to provide a general understanding of the overarching purpose to the Bluestone DSMS plan formulation. The information from this section serves as the basis for the risk assessment presented in Chapter 4. The USACE Dam Safety Planning process was used in selecting a risk management plan for this study. The six planning steps listed below are discussed in detail throughout the remainder of the report.

- Step 1: Identify Dam Safety Issues and Risk Reduction Opportunities (Chapter 3)
- Step 2: Estimate Existing and Future without Federal Action Condition Risk (Chapter 3 discusses assumptions & Chapter 4 includes the ECRA & FWAC risk assessment results)
- Step 3: Formulate Alternative Risk Management Plans (Chapter 5)
- Step 4: Evaluate Alternative Risk Management Plans (Chapter 5)
- Step 5: Compare Alternative Risk Management Plans (Chapter 5)
- Step 6: Select a Risk Management Plan (Chapter 5 and 6)

3.1. Dam Safety Issues, Opportunities, Objectives and Considerations

3.1.1. Dam Safety Issues (Problem Statement)

Two quantitative risk assessments were completed that describe the existing and future incremental risk of Bluestone Dam. These risk assessments are referred to as the existing condition risk assessment (ECRA) and the future without federal action condition (FWAC) risk assessment. The focus of these risk assessments are to define and identify PFM’s that present risk above the TRG. Other PFMs may warrant action in the context of ALARP principals and essential engineering guidelines if cost effective risk reduction measures can be identified. The total risk associated with these failure modes was quantified and found to be above guidelines for Average Annual Life Loss (AALL) and/or Annual Probability of Failure (APF). The ECRA and FWAC risk assessments assumed all work through Phase 4 of the DSA Project is complete. The primary conclusion from the ECRA and FWAC risk assessments are that PFM 33, Spillway Monolith Instability is the only failure mode driving total risk above the TRG. Two other PFMs were analyzed in detail, PFM34 (non-overflow monolith instability) and PFM 35 (abutment monolith instability) and both were determined to be below TRG. However, taking into account uncertainty, they were evaluated for additional risk reduction opportunities. These failures modes are described in detail in Chapter 4.

Step one in the plan formulation process is to identify the dam safety issues and risk-reduction opportunities. Based on the dam safety issues identified during the ECRA and FWAC risk assessments, the team developed a general issue statement with respect to Bluestone Dam and its associated risks.

“Upon completion of Phase 4 DSA construction activities at Bluestone Dam, there is incremental risk that remains from potential breach of the dam that are above USACE defined Tolerable Risk Guidelines.”

In summary the following list describes the dam safety issues at Bluestone Dam:

- Scour within the primary spillway stilling basin resulting from extreme floods has the potential to destabilize the overflow concrete gravity dam sections potentially leading to breach of the dam.
• Dam failure would likely result in significant consequences (loss-of-life, economic damages, and impacts to the human and natural environment).

3.1.2. Dam Safety Opportunities
The second part of step one in the DSMS planning process is to define the opportunities which exist as a result of developing alternative RMPs to address the dam safety issues. The following list describes the opportunities identified:

• Implement structural and/or non-structural measures to reduce incremental risk associated with dam failure and its consequences to achieve USACE tolerable risk guidelines.
• Identify the potential for other studies and/or actions that could be taken to reduce non-breach flood risks.

3.1.3. Dam Safety Objectives
The objective of this DSMS is to identify and recommend a cost effective RMP that supports the expeditious reduction of incremental risk. The incremental risk is the risk (likelihood and consequences) to the pool area and downstream floodplain occupants associated with the presence of the dam that can be attributed to breach prior or subsequent to overtopping, or component malfunction or misoperation. Non-Breach Risk is due to normal operation of the dam or overtopping of the dam without breach scenarios. Therefore, incremental risk is the difference in total risk in its current state (with dam safety issues and deficiencies) and risks assuming the dam functions as intended without dam failure (referred to as non-breach risk).

\[
\text{Incremental Risk} = (\text{Risk associated with the project in current state}) - (\text{risk associated with project without breach})
\]

Dam failure (breach) risk estimates require a quantification of the likelihood of the loadings, the likelihood of the structural response of the dam given the loading, and the adverse consequences (loss of life, property damage or lost benefits) given that failure occurs. As previously stated, for the purpose of this report, failure (breach) is defined as a sudden, rapid, and uncontrolled release of impounded water. When managing incremental risk for Federal dams, it is important to balance available financial resources while ensuring that citizens are provided a tolerable level of life safety.

The target for incremental risk reduction related to Bluestone Dam is to reduce risk to within USACE tolerable risk guidelines for annual probability of failure (APF) and Average Annual Life Loss (AALL), and when these guidelines are met, to consider additional opportunities to reduce risk to As Low as Reasonably Practicable (ALARP) as defined by Paragraph 5.3.8 of ER 1110-2-1156. The reduction in life safety risk (AALL) is considered to be of paramount importance. Two primary numerical values employed in this study to gauge the incremental risk at Bluestone Dam are APF and AALL. The following equations are used to compute APF and AALL and are integrated for the range of potential loading:

\[
\text{Annual Probability of Failure (APF) } = (Probability \text{ of Loading}) \times (Probability \text{ of Failure given the Loading})
\]

and

\[
\text{Average Annualized Life Loss (AALL) } = \text{(APF) } \times (Expected \text{ Loss of Life given the Failure})
\]
Where:

- **Probability of Loading** is the annual probability that the chosen load or load range will occur.
- **Probability of Failure given the Loading** is the probability that the dam will fail under the specific load or load range. This value is commonly referred to as System Response Probability (SRP) throughout the risk assessments.

U.S. Army Corps of Engineers’ (USACE) Tolerable Risk Guidelines (TRG) dictate that APF greater than or equal to 1 in 10,000 (0.0001 or 1 x 10⁻⁴) chance of breach per year is unacceptable except in extraordinary circumstances. The basis to take action to reduce or better define risk increases as the estimates become greater than 1 x 10⁻⁴ per year. The basis to take action to reduce or better define the risk diminishes as the estimates become smaller than 1 x 10⁻⁴ per year. ALARP considerations are also used to evaluate how far to reduce the APF below the guideline.

Life Safety (AALL) - The policy for the estimated AALL under USACE tolerable risk guidelines states as AALL further exceeds 1 in 1,000 (0.001 or 1 x 10⁻³) lives per year there is increasing justification to invest in risk reduction (i.e. life safety risk above 1 x 10⁻³ is generally considered unacceptable for the incremental risk between breach and non-breach except in extraordinary circumstances). Likewise, the basis to take action to reduce or better define the risk decreases as the estimates become lower than 0.001. AALL less than 1 x 10⁻³ is considered to be tolerable provided the other tolerable risk guidelines are met. ALARP considerations are used to evaluate how far to reduce risks below the tolerable risk limits until such actions are impractical or not cost effective. See Figure 3-1 for an illustration of the individual and societal guidelines.

Other risks defined in Engineering Regulation (ER) 1110-2-1156 include economic and environmental risk. These risks are defined in this report, however, specific tolerable risk guidelines do not exist for evaluation of these risks. Details of the existing condition environmental consequences are detailed in the Supplemental Environmental Impact Statement (SEIS) (Appendix K) and in Section 3.2.2.10. Environmental consequences of the selected plan can be found in Section 6.9. Economic consequences can be found in Section 4.5.3 and Appendix J.

In summary, the overall purpose of the study is to identify appropriate actions to manage risk. The objective of the study is to recommend a risk management plan that will reduce incremental risk to satisfy USACE tolerable risk guidelines. Additional objectives include: comply with essential USACE guidelines to the extent that it enables cost effective incremental risk reduction; incorporate ALARP principles where feasibly practicable; and comply with all applicable laws, regulations and guidelines.
Figure 3-1: Individual (left) and Societal (right) Guidelines for Incremental Risk

3.1.4. Risk Reduction Planning Considerations

There are also project considerations which, similar to constraints guide plan formulation and that the planning study will consider, but they are not hard constraints. The following list defines the planning considerations identified specific to the Bluestone DSMS process:

To minimize:

- impacts to authorized project purposes
- interim increase in risk during implementation (construction) of a risk management plan
- significant impacts to the environment
- impacts to high value aquatic habitats downstream of Bluestone Dam
- impacts on recreation activities
- social and economic impacts

3.2. Existing and Future without Project Condition Assumptions

The U.S. Water Resources Council’s Principles and Guidelines provide the instructions and rules for Federal water resources planning, one of them being to compare the effects of alternative plans based on the most likely future conditions with and without those plans in place. To accomplish this, descriptions (often called forecasts) must be developed for two different future conditions: the future without Federal Action condition (FWAC) and the future with project condition. The FWAC describes what is assumed to be in place if none of the study’s alternative plans are implemented. The differences between the FWAC (No Action) Plan and the future with project conditions are the effects of each plan.
The existing condition characterization and risk assessment for Bluestone Dam was based on the satisfactory completion of the Phase 3 and Phase 4 construction contracts of the DSA Project, as well as completion of the remaining designed anchors for the main dam structure. Additional information associated with the remaining anchors to be installed can be found in Section 3.2.2.1.4 (Existing Anchors Section).

The FWAC describes what is assumed to be in place if none of the study’s alternative plans are implemented by the Federal government. The FWAC is the same as the FWAC (No Action) Plan that is required to be considered by Federal regulations implementing the NEPA of 1969.

Table 3-1 summarizes the different assumptions that were used for the ECRA and FWAC Risk Assessments. Mobilization rate changes are discussed in Section 3.2.3.9.

Contract actions that are required to meet the FWAC include the installation of the approximate 66 anchors and remaining mitigation commitments as defined in the 1998 EIS. Rehabilitation of crest gate machinery is also required as FWAC and is likely to be a combination of in house maintenance and contract actions.

Table 3-1: Existing and Future Condition Assumptions for the Risk Assessments

<table>
<thead>
<tr>
<th>Assumptions</th>
<th>Existing Condition (ECRA)</th>
<th>Future Condition (FWAC)</th>
<th>Basis of Change</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hydrology</td>
<td>Overtopping Frequency = 1 in 100,000 to 1 in 10,000,000</td>
<td>Overtopping Frequency Best Estimate = 1 in 1,000,000</td>
<td>Estimated Depth of Overtopping at IDF = 18-ft</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>At top of dam there is no significant change between ECRA and FWAC condition for the inflow design flood (IDF) assumptions</td>
</tr>
<tr>
<td>Mobilization Rates for New and Kanawha Rivers (Best Estimates)</td>
<td>8 hour = 85%</td>
<td>8 hour = 93%</td>
<td>Given discussions with local and state emergency managers, they expect to implement a reasonable level of improvement in technology; emergency planning and response; and warning system capabilities between the ECRA and FWAC.</td>
</tr>
<tr>
<td>Water Control Manual and Dam Operations</td>
<td>Used original WCM dated 1994 with penstock gates operating after spillway gates are fully opened and prior to dam overtopping.</td>
<td>The revised WCM with a permanent threshold discharge in the primary stilling basin and includes penstock gates operating prior to spillway gates being fully opened.</td>
<td>Due to instability of the stilling basin weir at discharges above 140,000 cfs (threshold discharge), there is higher dam safety risk when the discharge exceeds this threshold. Therefore, it is implied there are less dam safety risk when the discharge is released through the penstock gates.</td>
</tr>
<tr>
<td>Population at Risk (PAR) from failure for the Inflow Design Flood (IDF)</td>
<td>174,002</td>
<td>165,302</td>
<td>The ECRA used the 2014 Census data and the FWAC used population projections from the West Virginia University Bureau of Business and Economic Research. The future populations at risk is projected to decrease over the period of analysis.</td>
</tr>
<tr>
<td>DSA Approved Anchors</td>
<td>66 anchors in addition to what has been procured in Phase 4 DSA construction.</td>
<td>66 anchors in addition to what has been procured in Phase 4 DSA construction.</td>
<td>There is no change between these two assumptions. These anchors are required to realize the ECRA and FWAC risk results. The implementation cost are paid for by Construction General (CG) as opposed to O&amp;M funds.</td>
</tr>
<tr>
<td>Operation and Maintenance of Spillway Crest Gates (Reliability)</td>
<td>Replacement of all hoisting wire ropes and the right-angle gear reducers of gates number 7 and 9.</td>
<td>Assumes gate machinery will be rehabilitated and maintained to ensure reliable operation using O&amp;M program funding.</td>
<td>Responsible dam ownership duties to keep up with operation and maintenance activities.</td>
</tr>
</tbody>
</table>
3.2.1. Planning Horizon

The planning horizon encompasses the study, construction, and the post construction life cycle of the project. The time frame used when forecasting the future with and without project conditions while considering impacts of alternative plans is called the period of economic analysis. The period of analysis for water resources projects is typically between 50 and 100 years (U.S. Army Corps of Engineers, 2000). For this study, the period of analysis is 50 years due to the uncertainty in forecasting beyond that point. DSMS construction is expected to be completed in 2041, based on predicted funding stream and sequencing. Bluestone has multiple phases of construction currently underway for dam safety remediation under the DSA Program. Therefore, the period of analysis for the DSMS is considered to span FY 2019 to FY 2069. By incorporating a 50-year period of analysis to reflect beneficial and adverse effects of the project through time, the period of analysis for the proposed project will end in the year 2069. Figure 3-2 illustrates the period of analysis.

![Figure 3-2: Planning Horizon](image)

3.2.2. Existing Conditions

3.2.2.1. Existing Dam Description

The dam is a straight, concrete gravity structure with a maximum height of 165 feet above the stream channel. As previously stated, the length of the dam is 2,048 feet at EL 1,535-ft. For purpose of the existing condition description, the dam is divided into six distinct sections: left and right abutment sections, spillway section, assembly bay section, penstock section, and non-overflow section. Details of the existing dam can be found in Figure 3-6 through Figure 3-8.

See the following sections in Appendix C for detailed drawings of the existing dam:

- Original Dam As-Built Drawings in Section XVII;
- Drift and Debris Tower As-built Drawings in Section XVIII;
- Phase 1 As-Built Drawings in Section XIX;
- Phase 2A As-Built Drawings in Section XX;
- Phase 2B As-Built Drawings in Section XXI;
- Phase 3 Current Record Drawings in Section XXII; and
- Phase 4 Current Record Drawings in Section XXIII.
3.2.2.1.  Existing Left and Right Abutment Sections
The right and left abutment sections include Monoliths 1 through 8 and 45 through 55, respectively. Some modifications have been made on both abutments as part of the Phase 2A construction contract under the Dam Safety Assurance program.

3.2.2.1.2.  Existing Spillway Section
The spillway section, comprised of Monoliths 25 through 44, is located adjacent to the left abutment section and includes the gated spillway, sluiceways, spillway stilling basin, and service bridge. The gated concrete ogee spillway section is 790-ft wide, with a crest of EL 1,490-ft and is flanked by training walls. Spillway flow is controlled by 21 vertical lift gates. Each gate is 31 feet high and spans 30 feet horizontally between spillway bridge piers that are 8 feet wide. The service bridge consists of two reinforced concrete girders and spans the spillway section to support the gantry crane. There are also 16 gated sluices, 5-ft and 8-inches wide by 10-ft high, with inverts at EL 1,389-ft to regulate low flows. The stilling basin is a two stage basin which includes an apron located immediately downstream of the monoliths, followed by an unprotected length of rock floor, and then a downstream stilling weir that retains tailwater to help dissipate energy from spillway flows. This weir is 23 feet high and has a crest elevation of EL 1,391-ft and is located 364 feet downstream from the axis of the dam. The stilling weir ties into the downstream ends of the stilling basin training walls. Energy from spillway flow is dissipated by a double row of baffles on the apron of the dam (first stage) and a double row of baffles immediately below the stilling weir (second stage).

The original design discharge capacity of the dam is 430,000 cfs which is the same as the estimated peak inflow. The original hydrologic design for Bluestone Dam was based on a hypothetical flood created by shifting the center of the July 1916 hurricane storm to the New River drainage basin. This hypothetical flood served as the basis for the Spillway Design Flood (SDF). Normal operation of the reservoir is by 16 double gated sluices with a maximum total discharge capacity of 72,000 cfs (at pool EL 1,517-ft with no spillway crest gate flow).

Drift and debris problems necessitated construction of a gated tower structure to facilitate passage of drift and debris. This tower, constructed in 2005, includes a tunnel through monolith 35 that exits into the spillway stilling basin.

3.2.2.1.3.  Existing Assembly Bay Section
The assembly bay is essentially a four-story reinforced concrete structure on the downstream face of the dam. This section, consisting of monoliths 22, 23, and 24, contains the electrical and maintenance equipment. Monolith 24 contains the main office and a ten-foot wide trash chute. Original construction included the trash chute at EL 1,485-ft to pass drift and debris. Because hydropower was never implemented, normal pool was lowered from EL 1,490-ft to a summer pool of EL 1,410-ft and winter pool of EL 1,406-ft, rendering the trash chute ineffective. The trash chute was sealed by construction of a concrete wall as part of the Phase 2A contract.

3.2.2.1.4.  Existing Anchors
For the existing condition it is assumed that all rock anchors (~ 500) through phase 4 are installed in the dam structure. Other existing anchors include 240 bar anchors in the penstock stilling basin, as well as 112 strand anchors in the exit pad for the auxiliary penstock spillway.

Although not yet procured by the active DSA project, the ECRA (and the FWAC) assumes that an additional 66 +/- multi-strand rock anchors will be installed in the dam. As these anchors are required to meet the existing risks portrayed, these anchors will have to be installed in the FWAC (No Action) Plan. It should be
noted, these anchors are also included in many of the RMPs discussed in Chapter 4. The cost to install the anchors is estimated to be approximately $15M, but would likely cost more in the FWAC (No Action) Plan, because a contractor would have to mobilize and install a platform across the primary spillway just to install many of these anchors.

### 3.2.2.1.5. Existing Penstock Section and Auxiliary Penstock Spillway

This portion of the dam (monoliths 16 through 21) is a gravity section with six embedded penstocks that were originally intended for hydropower, but was never implemented. As part of the Dam Safety Assurance (DSA) effort, massive concrete thrust blocks were installed on the downstream face of monoliths 15 through 21. To convert the six penstocks into an auxiliary spillway, they were extended through the thrust blocks and fitted with hydraulically-operated bottom-hinged gates in order to increase overall discharge capacity. An auxiliary penstock stilling basin has been constructed downstream of the penstocks as a part of the DSA project and consists of concrete training walls, divider walls, anchored scour protection, baffle blocks, and an anchored exit pad. A downstream cofferdam was utilized to unwater and construct this basin. The cofferdam was left in place to support future construction efforts recommended from this study. The ECRA assumes a fully functional auxiliary penstock spillway with removal of the cofferdam for risk estimating purposes. However, it is anticipated that leaving the cofferdam in place has negligible effect on the conclusions of the ECRA.

### 3.2.2.1.6. Existing Non-overflow Section

This section lies between the right (east) abutment and penstock sections and includes monoliths 9 through 15. The inspection gallery extends through these monoliths, although the operation gallery does not.

### 3.2.2.1.7. Existing Foundation

Bluestone Dam is situated within a maturely dissected section of the Appalachian Plateau physiographic province. The dam is located in the New River Valley, which is cut from the shales and sandstones of the Hinton Formation, within the Mauch Chunk Group, of Upper Mississippian Age (see Paragraphs 1.1. Geologic History and 1.2. General Bedrock Geology, Appendix E).

The typical founding elevation of the dam is approximately EL 1,358-ft MSL where it rests on the Stoney Gap Sandstone, the basal member of the Hinton Formation. The Stoney Gap Sandstone forms the bed of the New River at the site and ranges from 52 to 100 feet thick below the present level of the stream. This sandstone is very hard, gray, dense, well-cemented siliceous quartzitic sandstone (called Orthoquartzite in later reports) with occasional thin interbeds and lenses of black shale (Interbedded Orthoquartzite and Shale). Bedding is essentially horizontal and often stylolitic in massive beds although there are locally cross-bedded zones with bedding planes dipping up to about 45°, and zones showing ripple-like features. The Stoney Gap Sandstone is underlain by a dense, moderately hard, maroon and gray claystone called the Coney Shale. The rocks forming the dam abutments consist of shales and siltstones interbedded with sandstones. The bedrock units at the site are somewhat lenticular, so that variations in rock types occur horizontally within individual units.

The regional dip for the area is about 3° or 4° to the south. Regional dip is interrupted at the dam site by the presence of the Bellepoint Syncline, the axis of which is aligned perpendicular to the dam axis and intersects the dam in the right abutment (see Paragraph 1.3. Geologic Structure, Appendix E). Bedrock in the abutments dips gently into the valley from the northwest and southeast. The beds in the valley bottom are nearly horizontal with a low angle dip, generally less than about 5°, to the south-southwest (upstream).
Vertical to near vertical joints are present throughout the foundation in the valley bottom and have been measured spaced as closely as two feet apart in outcrop and closer in portions of the foundation (Figure 3-3 and Figure 3-5, below). Figure 3-3 was produced by Borehole Camera Logging of Borings Drilled in 2011 and 2013. The primary joint set is oriented between $N66^\circ E$ and $N85^\circ E$, with a secondary set typically oriented $N39^\circ W$ to $N0^\circ W$ in the dam foundation. Downstream of the weir the secondary set ranges closer to approximately $N0^\circ E$ to $N30^\circ E$, approximately perpendicular to the dam axis (see Paragraph 1.4. Site Geology, Appendix E). Additionally, some fractures strike nearly parallel to the dam axis orientation of $N78^\circ W$ and slickensided fractures have been exposed downstream of the dam in the foundations of the penstock section thrust blocks and the penstock stilling basin with orientations of $N62^\circ W$ to $N65^\circ W$, dipping $24^\circ-25^\circ$ (upstream). Blocks that would be created by these joint sets are split into sheets by the closely spaced bedding (0.1’-0.2’) in the interbedded orthoquartzite and shale that makes up the river bed and foundation of the main stilling basin monoliths, creating a structure similar to a deck of cards that has significant bearing capacity, but reduced resistance to sliding and scour.

![Figure 3-3: Rose Diagram Showing Orientation of High Angle Fractures](image)

Two faults (see Figure 3-4 and Figure 3-5), sometimes referred to as gouge zones in early reports, have been identified at the project and create stability concerns for the main dam that may need addressed as part of the overall risk management solution. One fault was identified beneath monoliths 10 through 14 that varies in thickness from as little as a half inch up to approximately 3 feet, dipping about 10 degrees to the east with a strike of $N10^\circ W$. Seepage through the fault was inhibited by a cutoff wall constructed upstream of monoliths 10 through 13. At monolith 10, construction documents show that the fault appeared to pinch out, although it may continue to extend beneath the foundation of the abutment monoliths as a slickensided plane.
Figure 3-4: Intersection of the Faults beneath the Dam Axis (Offset Station 2+00)

The second fault is oriented with a strike of approximately N5°W and dips generally around 17 degrees to the west, and varies from approximately 0.1 to 3-feet thick. It is possibly present beneath portions of monoliths 13 and 14 where it was at least partially removed and daylights in the foundations of monoliths 15 through 17, extending deeper into the dam foundation toward the west (see Paragraph 1.4. Site Geology, Appendix E). This fault crosses the foundation of the auxiliary penstock stilling basin and reduces scour resistance of the rock downstream, however modeling and analysis has demonstrated that the basin will be stable for the design flows. The fault daylights well downstream of the main stilling basin and is not expected to impact construction or performance.

Figure 3-5: Daylight of the Fault (triangle area) through the Center of the Auxiliary Penstock Stilling Basin Foundation
Figure 3-6: Existing Dam Overview
Figure 3-7: As-Built Drawing of General Plan for Bluestone Dam
(The Non-Overflow section in this figure is what was evaluated for potential failure modes in the Baseline Risk Assessment. Please note the non-overflow section is larger than just this section. Please reference Figure 3-6 for the entire non-overflow section.)

Figure 3-8: Plan View of Dam Showing Monolith Numbers
3.2.2.1.8. **Existing Gates and Operating Equipment**

This section describes the FWAC assumptions for the existing sluice gates, crest gates, penstock gates, debris and trash gates, and the existing electrical equipment.

**Existing Sluice Gates**

Each of the 16 sluice conduits contains a service and an emergency gate that are hydraulically operated by manually opening valves in the operation gallery. The service and emergency gates are bonneted slide type gates. The gate invert is EL 1,389-ft.

The sluice gate HPUs are original equipment and have had no rehabilitations or upgrades since first going into operation. The gates are operated by manually opening three-way valves and then pushing a remote start button to provide flow to the cylinder. These manual valves are extremely hard to operate and have resulted in project personnel bending wrench handles while operating the valves. PI reports state that the hydraulic piping is leaking at several different flanged connections due to seal deterioration. Each gate is operated approximately 150 times per year.

Although there are significant issues with operation of the sluice gates, the relatively small discharge capacity with respect to the total discharge through the spillway results in negligible impact to the loading and response of the structure. For simplification in the risk estimate, all sluice gates are assumed to operate properly.

**Existing Crest Gates**

The machinery to open the 21 spillway lift gates (gates include roller trains) is motor-driven through a worm gear reducer, two gear reductions and then the wire rope drum which operates both wire ropes on either side of the vertical gate. The wire rope then operates as an eight part line via lower sheave nests attached to the gate and the upper sheave nests attached below the machinery. The top of gates are located at EL 1,521-ft with the spillway crest at EL 1,490-ft.

The gates are horizontally framed, consisting of nine girders, three vertical diaphragms, and a 5/8” skin plate, assembled in three sections with field welds at two of the girders. The girders are built-up members, seven of which are W-sections with riveted cover plates and the remaining two girders are built-up from angles and plates riveted together at the locations of the field welds. The attachment of the gate to the hoist assembly consists of two 5/8” pin plates on each side of the gate (four total) bolted to the top girder.

The 2014 PI report notes that the wire ropes had areas of corrosion and flat spots from wear which were also noted during the inspection for the Baseline Condition Risk Assessment (2013 BCRA) report. The intermediate gears and pinions were also noted to have a lot of wear, radial grooves, and pitting. As a result of PI's and 2013 BCRA recommendations all of the crest gate hoisting wire ropes have been replaced as well as two of the right-angle gear reducers (gates 7 and 9) have been rehabilitated. The gear reducers were rehabilitated due to failure during regular periodic testing operations. The remainder of the crest gate machinery is original.

The reliability analysis, as described in Appendix A.5 of the Risk Assessment Technical Summary Report (US Army Corps of Engineers, Bluestone Dam Risk Assessment Technical Summary Report, 2016), indicates the probability of a single gate failing to open in their existing condition is 9.5%. This results in a 55% probability that two to four gates will fail to operate on demand. Therefore, the risks were calculated for a scenario of three gates failing to open and compared to risks for a scenario of all gates operating to show the effect of gate reliability on total project risk.
Existing Penstock Gates
Three of the six penstock gates were installed in Phase 1 and the remaining three as part of Phase 2B. The gates may be damaged after opening due to impact or high velocity flow and remain closed by utilizing an over center linkage, but also have a pin to prevent opening by vandals. Because the penstocks gates may be severely damaged during use, the open penstocks may not be readily closed requiring design, procurement, and placement of bulkheads after the pool drops. In the event that penstock gate openings are required, the bolts (6 per penstock) must first be removed then the hydraulic cylinder actuator will push the lever thereby unlocking the gate which will be opened by the force of the water inside the penstock. Hydraulic fluid flow is supplied to the actuator from an HPU when the manual valve is opened. The HPU and manual valves are located in the machinery room on the fourth floor of monolith 22 and at the penstock gates. The valves at the gates must be set in the correct position prior to tailwater preventing access to them. In the event of failure of the hydraulic system, manual operation of these gates would be hazardous during low tailwater events, but would be impossible during high tailwater events. The maximum estimated flow through one penstock during an extreme event is approximately 25,000 cfs (at pool EL 1,520-ft).

Given this is a newly designed and constructed feature it is assumed to be 100% reliable for all risk estimates.

Existing Debris and Trash Gates
The nine drift and debris tower gates were installed in 2005. These gates vary in size and are hydraulic cylinder-operated. The cylinder rods operating the drift and debris tower gates are ceramic coated and nearly always extended as the gates are only operated four times per year to pass drift and debris and otherwise remain closed. The HPU is located on the fourth floor machinery room in monolith 22. The debris gate HPU is in very good condition.

Existing Electrical Equipment
The Motor Control Center (MCC), located on the fourth floor machinery room of monolith 22, has been replaced since original construction (2014). The main line-side wiring going to the MCC was replaced in 2005 when incoming power was relocated on the project from the east abutment to the west abutment. However, all load side cabling leaving the MCC going to critical dam equipment is original. Five of the 21 crest gate power cables from the controller cabinet to the motor have been replaced in the last year due to insulation deterioration in the 60-plus year old cabling. This deterioration of insulation will continue until the remaining power and control cables are replaced. There is no record that any of the motors have been rehabilitated nor have the brake actuators. In 1984, the crest gate controllers were replaced.

3.2.2.2. Existing Dam Operations
Under the existing condition, Bluestone Dam is operated to maintain a winter pool at EL 1,406-ft from December through March and a summer pool at EL 1,410-ft from April through November. Minimum outflows of 610 cfs are required at all times. When downstream stages are projected to increase above control stages at either Hinton, Kanawha Falls, Charleston, or Point Pleasant, West Virginia based on observed rainfall, the project reduces project outflows (often in conjunction with Summersville and Sutton Dams) to store flood waters, which are then released as river elevations recede to prevent or minimize economic damages.

Prior to the current WCP deviation described in the next section, the project is operated to store flood flows until the forecasted inflow is of such a magnitude that is storing it in its entirety would result in a pool elevation greater than the Maximum Flood Control Pool (MFCP) of EL 1,520-ft. When it is projected that MFCP would be exceeded, the project is operated to release the projected excess storage above
MFCP in a manner that will minimize downstream flooding. Once inflows exceed the capacity of the spillway, gates are operated to the fully opened position until the pool falls below EL 1,520-ft. As designed, the Bluestone flood control pool has a storage of 592,600 acre-feet from EL 1,410-ft (summer pool) to EL 1,520-ft (flood control pool). A conservation pool exists between EL 1,390-ft (1-ft above invert of the sluice gate) and EL 1,406-ft (winter pool). These pertinent elevations are shown in the reservoir diagram (Figure 2-3).

The control stage at Hinton corresponds to about 90,000 cfs flow from the New and Greenbrier Rivers. The sluices are used first to control and pass outflow during normal to moderate flood events. The spillway gates are second in the operational sequence, and are used to pass flow during larger flood events in addition to the sluices. The penstocks are controlled by bottom hinged gates and cannot be closed after being opened. The penstocks are intended to be used only when forecasts for extremely large and rare flood events, such as the PMF or IDF, indicates that the dam could be overtopped without the use of the penstocks.

### 3.2.2.2.1 Existing Water Control Plan Deviations

On April 7, 2016, LRH requested a major deviation for the Bluestone Lake, Water Control Plan (WCP). On April 25, 2016, the deviation to the WCP (water years 2016 - 2023) was approved to reduce the risk of structural failure (breach) of Bluestone Dam. The deviation established a threshold flood that results in a pool of EL 1,510-ft (10 feet below the flood control pool) and a discharge of 140,000 cfs (290,000 cfs below maximum design discharge) through the primary spillway. The threshold flood is defined as the flood that fully (safely) utilizes the existing dam. To achieve this threshold, when the pool is forecast to exceed EL 1,510-ft the project will be operated to release flows through the sluice and spillway crest gates, up to 140,000 cfs, in an attempt to hold the pool at EL 1,510-ft. If inflow is great enough that this is not possible, the lake will be allowed to rise up to the top of crest gates upon which time the crest gates will be opened to prevent overtopping. When the pool is projected to approach EL 1,510-ft, downstream stakeholders will be notified. The deviation expires on 31 December 2019.

A potential amendment is forthcoming to the current deviation for water years 2016 - 2019 once a detailed consequence analysis is completed for the project (expected in FY17). The analysis will provide additional information to aid in decision-making on any additional optimization to the current deviation to balance the risks associated with structural failure and risk associated with increased discharge. If the analysis indicates viable opportunities to optimize these tradeoffs, then an amendment will be sought at that time.

The auxiliary penstock stilling basin became operational after Phase 3 construction was completed. The operational use of the six sacrificial penstock gates will need to be incorporated into the water control plan. This incorporation will initially be accomplished within the deviation amendment mentioned in the previous paragraph and secondarily be accomplished by a Water Control Manual update post Phase 4. For details of the timeline of WCP Activities see Table 6-2 and Figure 6-15.

The ECRA was finalized prior to approval of deviation water years 2016 - 2023, and uses the existing water control plan without deviation for the quantitative risk assessment. However, it does account for the penstock operation (penstocks open prior to dam overtopping at EL 1,535-ft) which is not included in the water control plan or its current deviation. Table 3-2 displays the primary differences between the different water control deviations and the ECRA.
Table 3-2: Existing Condition Assumptions for the Water Control Deviation Operations

<table>
<thead>
<tr>
<th>Current Deviation</th>
<th>Restricts discharge to 140,000cfs and restricts pool by 10-ft decrease from the maximum flood control pool.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Potential Amendment to Deviation</td>
<td>Defines pool and discharge restriction based on Consequence Study results. Will incorporate penstock operations.</td>
</tr>
<tr>
<td>ECRA Water Control Assumptions</td>
<td>Existing Water Control plan operations without deviation plus the penstock operation capacity</td>
</tr>
</tbody>
</table>

3.2.2.3. Existing Operation and Maintenance
The total annual cost of operating and maintaining Bluestone Dam is estimated to be around $1.73M, including labor, utility, and routine maintenance, engineering, environmental, recreational support activities and maintenance packages. The cost could vary depending on the amount of engineering support labor for significant flood events and the nature of required maintenance packages. The $1.74M does not capture life cycle cost associated with repair, replacement, and rehabilitation but reflects the FWAC. Specific details of the total annual O&M cost breakdown can be found in Table 6-6.

3.2.2.4. Existing Hydrology

3.2.2.4.1. Probable Maximum Precipitation
The PMP, IDF, and hydrologic loading curve have been updated. A site-specific PMP was conducted by USACE and resulted in an approximately 10% reduction in 72-hour rainfall depth as compared to the 1982 study estimate, and reported in the 1998 DSA and 2013 BCRA. A total PMP depth of 17.9-inches was estimated and the spatial distribution of total rainfall is illustrated in Figure 3-9. See Appendix D for additional information.

3.2.2.4.2. Inflow Design Flood (IDF)
The detailed IDF analysis was performed including the results of the updated PMP analysis. This update demonstrated a significant change in basin rainfall-runoff analysis. The main contributing factors for the IDF increase was accounted for in the change in sub-basin parameters within the hydrologic model, and the change in hydrologic routing for the reaches connecting the subbasins to the dam. The change in sub-basin runoff parameters was a result of the 2014 study using higher resolution data (hourly as opposed to 6-hour increment) and more intense calibration storm events that were recorded after the 1982 study was completed. The change in sub-basin parameters accounted for approximately 35% of the total increase in the IDF between the 1982 and 2014 studies. The change in hydrologic routing was a result of model calibration and the use of a physically based routing method (Muskingum-Cunge) that was calibrated and verified at stream gages throughout the watershed. The previous study utilized the Muskingum routing method which doesn't accurately calculate routing attenuation for discharges greater than the event used for calibration. The change in hydrologic routing method from Muskingum to Muskingum-Cunge accounted for approximately 65% of the total increase in the IDF between the 1982 and 2014 studies. The IDF update resulted in an approximately 35% increase over the DSA/BCRA peak...
inflow estimate. The adopted inflow design flood hydrograph had an estimated peak inflow of 1,463,400 cfs, as shown in Figure 3-10. See Appendix D for additional information.

The resulting routing of the IDF under the existing conditions results in a maximum reservoir EL 1,553-ft, 17-ft above the existing top of dam including curb. This routing assumes an antecedent pool of top of active storage (EL 1,520–ft) with operations based on the current WCP without deviation. The results of the IDF routing are illustrated in Figure 3-11. See Appendix D for additional information.

### 3.2.2.4.3. Hydrologic Hazard Curve

The hydrologic hazard curve was also updated. This update included several modified methods used for evaluation as compared to the BCRA loading curve; and incorporates stage-frequency relationships up to the IDF. The Flood Risk Analysis compute option within the Watershed Analysis Tool (HEC-WAT), along with a Hydrologic Engineering Center (HEC) Reservoir System Simulation (HEC-ResSim) model of the Kanawha watershed, were used to sample flood events and initial conditions and then route the flows through the HEC-ResSim model to define the Bluestone Dam pool stage frequency curve. The uncertainty analysis developed for the loading curve concluded that the largest impact to the results was dominated by the uncertainty in the volume frequency function based on the number of historic observations. The hydrologic hazard curve is illustrated in Figure 3-12 and additional information can be obtained in Appendix D.

The major parameters considered in the development of the hydrologic hazard curve include:

1) Flow-Frequency  
2) Starting Pool Elevation  
3) Hydrograph Shape  
4) Critical Flow Duration  
5) Penstock Operations

Of these parameters, it was determined through sensitivity analysis that the most influential parameter is the uncertainty associated with flow-frequency. The uncertainty associated with the flow-frequency curve ranges several orders of magnitude, compared to less than half order of magnitude of influence on the stage-frequency curve associated with other parameter sensitivities. A more complete discussion of uncertainty associated with the hydrologic hazard curve is contained in Appendix D.
Figure 3-9: 72-Hour PMP Rainfall Depth
Bluestone Dam – Final DSMR

Figure 3-10: Bluestone Dam PMF Inflow Hydrographs

- **Non-Peaked IDF**
- **125% Peaked IDF**
- **138% Peaked IDF**
- **150% Peaked IDF**

- **Discharge (cfs)**
  - **1,501,900 cfs (150%)**
  - **1,413,400 cfs (125%)**
  - **1,463,400 cfs (138%)**
  - **1,285,900 cfs**

- **Time (hours)**
  - 0, 24, 48, 72, 96, 120

Average Rainfall: 17.9 inches
Losses: 5.6 inches
Runoff: 12.3 inches
Figure 3-11: Bluestone Dam Level-Pool PMF Routing
Reservoir routing of synthetic events accomplished utilizing HEC-HMS. Coincidental probability was utilized to represent uncertain parameters (Inflow Hydrograph Shape and Starting Reservoir Storage).

Best Estimate stages below 1520 are defined by Plotting Positions of observed pools and routing of pre-project flood events (60 systematic pools and 3 Historic Pools developed by Routing Historic Flood Events).

Upper and Lower Bounds below a stage of 1520 were developed based on Ordered Statistics of observed pools and an equivalent POR of 68-Years.

Flow Probability is based on Log Pearson Type III distribution fit to 80 systematic years of record, 2-historic events, a historic period of record of 165-years, and an equivalent period of record of 82-years.

Flow frequency curve developed through Monte Carlo Simulation in the R Statistical Package.

The Best Estimate Curve represents Expected Probability computed through Monte Carlo simulation. The expected probability curve produced by pooling all realizations.

Upper Bound above 1520 is defined by routing scaled hydrographs based on the 95" Exceedance Probability Flows.

Existing Conditions Operations assume that the structure operates per the approved Water Control Plan, the Penstocks are operated at Top-of-Dam.

FWAC Operations assume that the structure maintains a 140 kcfs limit on discharge until the gates are at-risk of overtopping, at which point, the Penstocks are operated and a maximum induced surcharge curve is followed to Top-of-Dam.

Figure 3-12: Bluestone Dam ECRA and FWAC Hydrologic Hazard Curve
3.2.2.5. Existing Population at Risk
Immediately downstream of the dam is the Hinton, West Virginia, a community of approximately 2,500 people. The downtown area of Hinton is on the National Register of Historic Places. A dam failure will cause severe flooding in Hinton, particularly in the community of Bellepoint just below the dam, when failure occurs at pools above EL 1,434-ft. However, any prediction of significant spillway releases would trigger warnings and evacuations in the at-risk areas near the river.

Downstream of Hinton the river enters the New River Gorge, a steep and narrow gorge that has little to no riverside development. This extent of the river is known for white water rafting, fishing, and other outdoor recreation. It is not until the New and Gauley Rivers converge to form the Kanawha River approximately 65 miles below the dam that there is significant development near or within the inundation area. There are several small communities along the Kanawha River in the 65 - 85 mile reach below the dam, with more intensive development starting at Marmet, West Virginia. This increase in development continues downstream to the City of Charleston, West Virginia, located 104 miles downstream of the dam. Charleston, West Virginia is the largest at-risk population center in a dam failure scenario (Figure 3-13 and Figure 3-14). Charleston is the capital of West Virginia with a population of 50,404 (2014 US Census). The total population in the surrounding metropolitan area including suburbs is 304,200. The city is located in Kanawha County at the confluence of the Elk and Kanawha Rivers, approximately 58 miles upstream of the confluence of the Kanawha and Ohio Rivers. To give a better perspective of the downtown area, there are 21 buildings greater than ten stories tall, the tallest being 30 stories. The majority of the city is situated in the historic floodplain area of the river valley, approximately one mile wide with steep hills (200 -ft to 300-ft) bordering it on both sides. Communities that have developed in the Kanawha River valley include Marmet, Kanawha City, South Charleston, Dunbar, Institute, St. Albans, Nitro, and Poca, West Virginia. Interstates 64, 77, and 79 intersect in downtown Charleston. Industries located within and around the city have access to barge traffic via the Kanawha River and its navigation pools, which are provided by three USACE lock and dam projects. The Norfolk-Southern and CSX railroads run parallel to the river, including a portion of track utilized by the Amtrak passenger line that runs between Washington, District of Columbia and Chicago, Illinois.

Downstream of the more densely populated Charleston area the Kanawha Valley is comprised mostly of light residential and agricultural development until reaching the Ohio River valley. The Kanawha River empties into the Ohio River at Point Pleasant, West Virginia roughly 58 miles downstream of Charleston.

Charleston will be severely impacted by flooding associated with both breach and non-breach scenarios at infrequent or extreme flood events. Failures were simulated at the lower pools of EL 1,411-ft and EL 1,434-ft, but they did not cause the river to approach flood stage at Charleston. A failure at pool EL 1,494-ft is estimated to result in the Kanawha River at Charleston’s south side bridge gauge at approximately 17 feet above flood stage, nearly equivalent to the 1840 flood of record at that location. A failure at pool EL 1,520-ft would increase the river stage an additional 14-ft, putting the Kanawha River at nearly 31 feet above flood stage. Both breach and non-breach inundation at any of these higher elevations would cause devastating floods along the Kanawha River valley. Because most of the development is within the valley itself, this inundation would impact the majority of the populated areas.
Figure 3-13: IDF Inundation (Dam Failure Scenario) at Hinton, West Virginia and Charleston, West Virginia

Figure 3-14: Top Left: Overlook of downtown Charleston; Top Right: John E. Amos power plant; bottom: Riverside view of Charleston
Figure 3-15: Water Elevation at Charleston, West Virginia (Failures were initiated at different times in the RAS model, so the starting hour of zero is not relevant)
Figure 3-15 is a representation of a stage graph at the South Side Bridge in Charleston (gauge data obtained from NWS river gauges [http://water.weather.gov/ahps/]). An event resulting in the pool reaching EL 1,553-ft would result in high consequences regardless of dam failure. Note how significant the flooding would be prior to (and regardless of) the breach event flow. When attempting to characterize how populations would respond to various events, it is important to clarify that significant non-breach outflows from the primary spillway will create the initial impetus for warning and mobilizations; that response will occur prior to a breach scenario, and the breach flows will add additional depth of inundation to the non-breach flooding. Figure 3-16 shows a before and after visualization of a major flood event in Charleston with a magnitude similar to the EL 1,553-ft event; breach and non-breach events would both look similar from this perspective.

![Figure 3-16: Before and after Google Earth visualization of a major flood in Charleston, West Virginia](image)

The Ohio River stage would also be affected by the higher pool elevation failures. The floodwall at Point Pleasant, West Virginia (top EL 580-ft) would be at risk of being overtopped if the dam failed at a pool elevation significantly greater than EL 1,520-ft. A failure at EL 1,535-ft or EL 1,542-ft would overtop it by approximately one foot, and would come within several feet of overtopping the floodwalls at Guyandotte, Huntington, and Ceredo-Kenova in West Virginia. Most of these projects have not experienced high loading levels, their performance under the loading conditions resulting from dam breach are uncertain. At both Point Pleasant and Huntington, West Virginia the EL 1,520-ft failure stage is below the levels reached during minor flooding (such as in the spring of 2011), but the EL 1,542-ft failure event would cause flooding roughly equivalent to the record floods of 1913 and 1937, both of which caused several hundred fatalities within the region (although communications were poor and there were no floodwalls at those times).

Another specific concern downstream is the presence of abandoned and active chemical manufacturing facilities. If impacted by flood water, these facilities may increase consequences not only to loss of life, but the health of the overall human environment during and following the flood. Due to model constraints and available information, life loss associated with chemical facilities being inundated is not included in the ECRA.

### 3.2.2.6. Existing Socioeconomics

Based on the 2009-2013 American Community Survey, approximately 39% of individuals within Hinton, West Virginia live below poverty levels compared to 17.9% in the state and 18% in Summers County. The median household income within Hinton, West Virginia is $24,488, which is substantially lower than the median household income of $41,576 for the entire state and $33,784 for Summers County.
According to the U.S. Census Bureau, Charleston, West Virginia had a population of 50,404 in 2014. The median household income in Charleston is $48,527 and approximately 18.4% of residents live below poverty levels. A large concentration of industrial infrastructure exists within the Charleston metropolitan area. The chemical industry alone supports approximately 22,500 jobs in Kanawha and Putnam Counties, which represents almost 60% of manufacturing jobs in the greater Kanawha Valley. Downstream of the densely populated Charleston area, the Kanawha Valley is comprised mostly of residential and agricultural development until the Kanawha River empties into the Ohio River at Point Pleasant, West Virginia.

Population and demographic characteristics for the study area within close proximity to the dam were obtained from the projected 2015 U.S. Census Bureau. County-specific information is provided in Figure 3-17. State-specific information is provided in Table 3-3.

![Image of population and demographic characteristics for counties upstream and immediately downstream of Bluestone Dam](image)

**Figure 3-17: Population and Demographic Characteristics for Counties Upstream and Immediately Downstream of Bluestone Dam (Data Source: United States Census Bureau, 2015)**

**Table 3-3: Population and Demographic Characteristics for Virginia and West Virginia**

<table>
<thead>
<tr>
<th>Category</th>
<th>West Virginia</th>
<th>Virginia</th>
</tr>
</thead>
<tbody>
<tr>
<td>Population</td>
<td>1,844,128</td>
<td>8,382,993</td>
</tr>
<tr>
<td>Median Income</td>
<td>$41,576</td>
<td>$64,792</td>
</tr>
<tr>
<td>High School Graduates</td>
<td>84.4%</td>
<td>87.9%</td>
</tr>
<tr>
<td>College Graduates</td>
<td>18.7%</td>
<td>35.8%</td>
</tr>
<tr>
<td>Percent Minority</td>
<td>6.2%</td>
<td>29.7%</td>
</tr>
<tr>
<td>Percent in Poverty</td>
<td>17.9%</td>
<td>11.2%</td>
</tr>
</tbody>
</table>
Summers and Mercer counties closely represent the demographic characteristics of West Virginia as a whole. Monroe County has a lower percentage of minorities while Raleigh County has a lower percentage of caucasian population than West Virginia. Giles County has a higher percentage of caucasian population than Virginia as a whole. African-American, Asian, and Hispanic or Latino populations are lower in Giles County than in the state of Virginia.

Since 2010, Summers County has seen a nearly 5% decrease in the population. Mercer County has seen the most change over the past few decades with nearly a 20% drop in the population since 1950. Giles County has maintained a consistent population since the 1960s and has only seen minor increases and decreases.

Total employment for the Kanawha River valley peaked in about 1950, and then declined about 17% until 1970. National employment increased by 37% during this period. From 1970 to 1980, employment increased about 30%, which was above the growth rate for both West Virginia and the nation. From 1980 to 1990, however, employment decreased by about 8%. During the same period, the State employment decreased 3% while the national average increased 18%. The leading employment categories in 1990 were services, wholesale and retail trades, mineral extraction, and manufacturing.

From 2010-2014, the total population in the labor force for the nation was at 63.5%. Virginia had 64.8% of the population in the work force and West Virginia had 54.2% of the population in the workforce. From 2010 to 2014, the U.S. had 14.8% of the population living in poverty. Virginia had 11.2% and West Virginia had 17.9% of the population living in poverty.

### 3.2.2.7. Existing Warning Systems

There are several warning systems in the downstream area, with each county having a different level of development on their systems. Immediately downstream of the dam the communities of Bellepoint and Hinton do not have an emergency warning system but have been planning toward installation of a system. As of April 2017, the City of Hinton has purchased and installed a flood/emergency warning siren; however, it is not functional and will be connected and tested in the future. Currently, the residents of these communities would rely on warnings coordinated through the state emergency management and disseminated to the local area along with media and word of mouth. All the counties have the capability to issue warnings via Wireless Emergency Alert (WEA) and Emergency Alert System (EAS). Kanawha County has a reverse-911 system for landline phones and a subscription text alert system, although it takes longer to initiate the reverse-911 than it does other systems and it has a call capacity limited to a few thousand per hour. Kanawha and Putnam Counties have a siren warning system with over 50 sirens; more than 20 of those have voice capability. Raleigh County also has a reverse-911 system. These warning systems combined with television, radio, and social media communications create a fairly robust warning system. In January of 2014 a chemical spill on the Elk River rendered the water supply of Charleston unusable; once the decision was made to issue a do-not-use warning communications began within 15 minutes over all types of media. Emergency responders involved with the spill generally agree that the warning communication went smoothly and quickly once it began. However, the emergency responders in general have noted that the decision to issue the warning took too long because of the lack of information about the magnitude of the spill and the danger posed by the chemical.

### 3.2.2.8. Existing Evacuation Routes

Evacuation routes near the dam include Routes 3 and 20 leading to Beckley, Princeton, and I-64. These routes will become inundated as the reservoir elevation rises and spillway flows increase. If heavy rain has fallen in the area, other obstacles could block local roadways such as trees, rockslides, and landslides. Once vehicular access is cut off, evacuation would be limited to getting to higher vertical elevations via
vehicle or on foot. Primary evacuation routes from the Kanawha-Putnam areas include I-64 Westbound, I-77 Northbound, I-79 Northbound and Route 119 (also called Corridor G). Some of the interstates have electronic signage that could be used to provide instructions, but there is no plan and likely not enough resources to implement a contra-flow traffic pattern on any of the major evacuation routes. Once flooding begins in the Charleston area, access ramps onto the interstate system will be flooded fairly quickly. Another consideration is that it will be difficult to find ideal places to evacuate to. Evacuees would need to travel several hours’ north on I-77 or I-79 to get to a larger city, or west along the Charleston-Huntington corridor of I-64 to reach areas with enough infrastructure to accommodate displaced populations.

The Yeager Airport in Charleston would be safe from flooding and may serve as an emergency evacuation point prior to flooding arrival, but access roads to the airport would be flooded by breach inundation making ingress and egress by anything other than air difficult.

3.2.2.9. Existing Public and Stakeholder Awareness

Public awareness for a potential dam breach is moderate. There has been ongoing construction at the project site since 2000. Over the years there have been multiple public and stakeholder awareness meetings throughout the inundation area. State and local governments, as well as the various chemical industries in the valley, should be reasonably aware of the risks because of these meetings. In addition to stakeholder meetings, emergency exercises have been performed with involvement from multiple levels of government. The level of general public awareness varies by location. The citizens in areas near the dam, primarily in the Hinton area, are well aware of the dam, its operations, and the potential risks during heavy flooding. In the Kanawha-Putnam area, where the dense population generally leads to higher risk, the public is not as aware of the dam due to the fact that it is so far away from them. During the 2016 flooding when Summersville Dam (also upstream of Charleston on the Gauley River) held back a significant amount of water, news articles referred to it as “a dam you may not know about” even though it is generally more well-known than Bluestone Dam because it is closer and has more recreation associated with it.

Risk communication efforts have continued consistently since 2010; with regular coordination with state, county and local emergency planners and responders. A renewed focus on non-breach risk began in 2015 and will continue as an effort that will continue in perpetuity. A public service announcement and public educational tools are also being prepared to help the general public better understand their risks.

3.2.2.10. Existing Environment Resources and Consequences

In March of 2014, the U.S. Fish and Wildlife Service provided a Planning Aid Letter (PAL) for the Bluestone Dam study area (See Appendix K for the letter). The PAL provided existing condition information for the existing aquatic and terrestrial habitats. The habitats within many portions of the study area are of high quality. The intricate water system that comprises the Kanawha River Basin and Bluestone River Basin contains a high diversity of quality fish and wildlife habitats. The aquatic environment downstream of the Bluestone Dam has been designated as Resource Category 1 habitat, which according to the U.S. Fish and Wildlife Service Mitigation Policy are of high value and are unique and irreplaceable on a national basis or in the ecoregion. The tailwaters have numerous fish nursery areas in vegetated shallows of water willow, abundant fish food sources, and ample places for fish to seek shelter behind large rocks. Such high value riverine habitats are relatively scarce in the ecoregion. Additional details on the existing environment can be found in the SFEIS, Appendix K.

Another specific concern downstream is the presence of abandoned and active chemical manufacturing facilities. If impacted by flood water, the facilities may increase consequences not only to loss of life, but the health of the overall human environment during and following the flood. Strong societal concerns have been expressed by emergency planners and responders downstream about the secondary and
tertiary impacts associated with toxic spills or mixing of chemical compounds which can pose both immediate dangers and long-term hazards that may require extensive remediation. While not quantified, as it is difficult to estimate the impact from this issues and the chemical industries guard information concerning onsite materials, it is recognized that the presence of these facilities likely increases the consequences from flood water releases from Bluestone Dam.

3.2.2.10.1. Vegetation
The project area falls within the eastern deciduous forest biome (Yahner 2000), a biome which is dominated by deciduous trees – broad leaved trees which lose their foliage each winter. The vegetation ranges from old growth forests to wetlands, and is a function of habitat, elevation, geology, soil, moisture and human activities. Habitats are similar at similar elevations in the study area, and species assemblages vary depending on relative humidity, with mesic habitats having higher ground story species richness than xeric habitats (Perles 2010). The study area around the New River upstream and downstream of the Bluestone Dam is considered an ecotone of the northern boundary for southern plant species and the southern boundary for northern plant species, contributing to the area’s overall botanical diversity (NPS 2009a). While a majority of the area outside of the small towns is forested, a small portion has been developed into agricultural fields and pastures.

3.2.2.10.2. Wetlands
The SEIS study area for wetlands has been defined as a one-half mile corridor along either side of the New River from Narrows, Virginia to Pt. Pleasant, West Virginia, which correlates with the four reconnaissance areas utilized in the 1998 FEIS. This portion of the New River traverses through mountainous terrain of sandstone and shell bluffs and steep slopes. Wetlands within this area generally occur along the edges of the river/lake banks or on flats/islands within the floodplains. According to National Wetland Inventory (NWI) maps, there are 899 acres of wetland habitat, 2.5 acres of open water in the form of lake or pond habitat, and 15,220 acres of riverine habitat within the defined study area (USFWS 2016a). Wetland acreages are only estimates from aerial photo interpretation. A past study by the National Park Service (NPS) compared the data from NWI maps to an actual delineation along the New River from Hinton to the I-64 Bridge. Results showed a 35.5% increase of wetlands found during the wetland delineation over the data from the NWI maps. Therefore, actual wetland acreage within the corridor may be considerably greater than what is shown on the NWI maps. Though wetlands area are shown to occur in the study area, no wetlands have been identified within the Construction Work Limits (CWL) of the DSMS.

3.2.2.10.3. Threatened and Endangered Species
Twelve federally listed threatened and endangered species are known to occur within the study area. Eight state listed threatened and endangered species are known to occur within the project area. Table 3-4 displays Federal and state-listed threatened and endangered plant species in the study area. Table 3-5 displays Federal and state-listed threatened and endangered animal species. Table 3-6 displays the Federally-listed endangered aquatic species in the study area. Table 3-7 displays the West Virginia and Virginia State imperiled aquatic species of the Bluestone National Scenic River, New River Gorge National River, and the Gauley River National Recreation Area.
Table 3-4: Federal and State-Listed Threatened and Endangered Plant Species in Study Area

<table>
<thead>
<tr>
<th>Common Name</th>
<th>Scientific Name</th>
<th>Federal Status</th>
<th>State Status</th>
<th>Potential to Occur in Project Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>Running buffalo clover</td>
<td><em>Trifolium stoloniferum</em></td>
<td>Endangered</td>
<td>Endangered (WV, VA)</td>
<td>Possible upstream of Gauley Bridge; known population at mouth of New River at confluence of Kanawha</td>
</tr>
<tr>
<td>Virginia spiraea</td>
<td><em>Spiraea virginiana</em></td>
<td>Threatened</td>
<td>Threatened (WV, VA)</td>
<td>Upstream of Gauley Bridge</td>
</tr>
<tr>
<td>Peters mountain mallow</td>
<td><em>Iliamna corei</em></td>
<td>Endangered</td>
<td>(Endangered VA)</td>
<td>Single occurrence, Giles County, VA</td>
</tr>
<tr>
<td>Bentley’s coralroot</td>
<td><em>Corallorhiza bentleyi</em></td>
<td>n/a</td>
<td>Endangered (VA)</td>
<td>Known occurrences in Monroe County, WV</td>
</tr>
<tr>
<td>Long-stalked holly</td>
<td><em>Ilex collina</em></td>
<td>n/a</td>
<td>Endangered (VA)</td>
<td>Possibly present, High elevation wetlands</td>
</tr>
</tbody>
</table>

Table 3-5: Federal and State-Listed Threatened and Endangered Animal Species

<table>
<thead>
<tr>
<th>Common Name</th>
<th>Scientific Name</th>
<th>Federal Status</th>
<th>State Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Indiana bat</td>
<td><em>Myotis sodalis</em></td>
<td>Endangered</td>
<td>Endangered (WV)</td>
</tr>
<tr>
<td>Virginia big-eared bat</td>
<td><em>Corynorhinus townsendii virginianus</em></td>
<td>Endangered</td>
<td>Endangered (WV)</td>
</tr>
<tr>
<td>Northern long-eared bat</td>
<td><em>Myotis septentrionalis</em></td>
<td>Threatened</td>
<td>None</td>
</tr>
<tr>
<td>Peregrine falcon</td>
<td><em>Falco peregrinus</em></td>
<td>None</td>
<td>Threatened (VA)</td>
</tr>
</tbody>
</table>

Table 3-6: Federally-Listed Endangered Aquatic Species in Study Area

<table>
<thead>
<tr>
<th>Taxa</th>
<th>Common Name</th>
<th>Scientific Name</th>
</tr>
</thead>
<tbody>
<tr>
<td>Invertebrate (mussel)</td>
<td>Fanshell</td>
<td><em>Cyprogenia stegaria</em></td>
</tr>
<tr>
<td>Invertebrate (mussel)</td>
<td>Pink mucket pearly mussel</td>
<td><em>Lampsilis abrupta</em></td>
</tr>
<tr>
<td>Invertebrate (mussel)</td>
<td>Sheepnose</td>
<td><em>Plethobasus cyphyus</em></td>
</tr>
<tr>
<td>Invertebrate (mussel)</td>
<td>Spectaclecase</td>
<td><em>Cumberlandia monodonta</em></td>
</tr>
<tr>
<td>Invertebrate (mussel)</td>
<td>Tuberculed blossom</td>
<td><em>Epioblasma torulosa torulosa</em></td>
</tr>
<tr>
<td>Fish</td>
<td>Diamond darter</td>
<td><em>Crystallaria cincotta</em></td>
</tr>
</tbody>
</table>

*may now be extinct (USFWS 2014).
Table 3-7: West Virginia and Virginia State Imperiled Aquatic Species of the Bluestone National Scenic River, New River Gorge National River, and the Gauley River National Recreation Area

<table>
<thead>
<tr>
<th>Taxa Category</th>
<th>Common Name</th>
<th>Scientific Name</th>
<th>State</th>
<th>State Ranking</th>
<th>Global Ranking</th>
<th>Park Occurrences</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fish</td>
<td>Ohio lamprey</td>
<td><em>Ichthyomyzon bdellium</em></td>
<td>WV</td>
<td>S2</td>
<td>G3G4</td>
<td>GR</td>
</tr>
<tr>
<td>Fish</td>
<td>Popeye shiner*</td>
<td><em>Notropis ariommus</em></td>
<td>WV</td>
<td>S2</td>
<td>G3</td>
<td>GR</td>
</tr>
<tr>
<td>Fish</td>
<td>Northern madtom</td>
<td><em>Noturus stigmosus</em></td>
<td>WV</td>
<td>S1</td>
<td>G3</td>
<td>GR</td>
</tr>
<tr>
<td>Fish</td>
<td>Candy darter</td>
<td><em>Etheostoma osburni</em></td>
<td>VA</td>
<td>S1</td>
<td>G3</td>
<td>NR</td>
</tr>
<tr>
<td>Crayfish</td>
<td>Elk River crayfish</td>
<td><em>Cambarus elkensi</em></td>
<td>WV</td>
<td>S1</td>
<td>G2</td>
<td>GR</td>
</tr>
<tr>
<td>Mussel</td>
<td>Elk toe</td>
<td><em>Alasmidonta marginata</em></td>
<td>WV</td>
<td>S2</td>
<td>G4</td>
<td>B, NR</td>
</tr>
<tr>
<td>Mussel</td>
<td>Purple wartyback</td>
<td><em>Cyclonaias tuberculata</em></td>
<td>WV</td>
<td>S1</td>
<td>G5</td>
<td>B, NR</td>
</tr>
<tr>
<td>Mussel</td>
<td>Yellow lampmussel</td>
<td><em>Lampsilis cariosa</em></td>
<td>WV</td>
<td>S1</td>
<td>G3G4</td>
<td>GR</td>
</tr>
<tr>
<td>Mussel</td>
<td>Wavy-rayed lampmussel</td>
<td><em>Lampsilis fasciola</em></td>
<td>WV</td>
<td>S1</td>
<td>G3G4</td>
<td>B, NR</td>
</tr>
<tr>
<td>Mussel</td>
<td>Pocketbook</td>
<td><em>Lampsilis ovata</em></td>
<td>WV</td>
<td>S1</td>
<td>G5</td>
<td>B, NR</td>
</tr>
<tr>
<td>Mussel</td>
<td>Green floater*</td>
<td><em>Lasmigona subviridis</em></td>
<td>WV,VA</td>
<td>S2</td>
<td>G3</td>
<td>B, GR, NR</td>
</tr>
<tr>
<td>Mussel</td>
<td>Mapleleaf</td>
<td><em>Quadrula</em></td>
<td>WV</td>
<td>S2</td>
<td>G5</td>
<td>NR</td>
</tr>
<tr>
<td>Mussel</td>
<td>Lilliput</td>
<td><em>Toxolasma parvus</em></td>
<td>WV</td>
<td>S2</td>
<td>G5</td>
<td>B, NR</td>
</tr>
<tr>
<td>Mussel</td>
<td>Pistolgrip</td>
<td><em>Tritogonia verrucosa</em></td>
<td>WV</td>
<td>S2</td>
<td>G4G5</td>
<td>B, NR</td>
</tr>
<tr>
<td>Mussel</td>
<td>Rainbow</td>
<td><em>Villosa iris</em></td>
<td>WV</td>
<td>S2</td>
<td>G5</td>
<td>BR</td>
</tr>
<tr>
<td>Mussel</td>
<td>Pocketbook</td>
<td><em>Lampsilis ovata</em></td>
<td>WV</td>
<td>S1</td>
<td>G5</td>
<td>B, NR</td>
</tr>
<tr>
<td>Mussel</td>
<td>Green floater*</td>
<td><em>Lasmigona subviridis</em></td>
<td>WV,VA</td>
<td>S2</td>
<td>G3</td>
<td>B, GR, NR</td>
</tr>
<tr>
<td>Mussel</td>
<td>Mapleleaf</td>
<td><em>Quadrula</em></td>
<td>WV</td>
<td>S2</td>
<td>G5</td>
<td>NR</td>
</tr>
<tr>
<td>Mussel</td>
<td>Lilliput</td>
<td><em>Toxolasma parvus</em></td>
<td>WV</td>
<td>S2</td>
<td>G5</td>
<td>B, NR</td>
</tr>
<tr>
<td>Mussel</td>
<td>Pistolgrip</td>
<td><em>Tritogonia verrucosa</em></td>
<td>WV</td>
<td>S2</td>
<td>G4G5</td>
<td>B, NR</td>
</tr>
<tr>
<td>Mussel</td>
<td>Rainbow</td>
<td><em>Villosa iris</em></td>
<td>WV</td>
<td>S2</td>
<td>G5</td>
<td>BR</td>
</tr>
<tr>
<td>Salamander</td>
<td>Eastern hellbender</td>
<td><em>Cryptobranchus alleganiensis</em></td>
<td>VA</td>
<td>S2</td>
<td>G3 G4</td>
<td>NR</td>
</tr>
</tbody>
</table>

*Petitioned for Federal listing as endangered or threatened. The Green floater is state-listed threatened in VA.

State Rankings:
S1 – Critically imperiled – At very high risk of extirpation from the state due to extreme rarity (often 5 or fewer populations), very steep declines or other factors
S2 – Imperiled – At high risk of extirpation from the state due to very restricted range, very few populations (often 20 or fewer) steep declines, or other factors
S3—Vulnerable – At moderate risk of extirpation from the state due to a restricted range, relatively few populations (often 80 or fewer), recent and widespread declines, or other factors

Global Rankings:
G1 – Critically Imperiled – At very high risk of extinction due to extreme rarity (often 5 or fewer populations), very steep declines, or other factors
G2 – Imperiled – At high risk of extinction due to very restricted range, very few populations (often 20 or fewer) steep declines, or other factors
G3 – Vulnerable – At moderate risk of extinction due to a restricted range, relatively few populations (often 80 or fewer), recent and widespread declines, or other factors
G4 – Apparently secure - Uncommon but not rare; some cause for long-term concern due to declines or other factors
G5 -- Secure – Common; widespread and abundant

Park Occurrences:
B--Bluestone National Scenic River
NR--New River Gorge National River
GR -- Gauley River National Recreation Area
3.2.2.11. **Existing Land Use**

Most of the land in the study area is covered with second growth forests. Timbering operations were originally developed to provide wood supports for use in the mines. Lumber is now used for the manufacture of furniture, pallets, and building materials. Although lumbering is increasing in the study area, no large volume of commercial activity is expected to develop.

Agricultural land within the basin is primarily concentrated in Putnam and Mason Counties along the Kanawha River. Following national and state trends, agriculture is declining in importance to the economy of the area. Prime farmland is located within the study area. The Natural Resources Conservation Service (NRCS) has determined that prime farmland will not be impacted as part of the project.

Most of the industry in the basin is concentrated in Kanawha County along the floodplain. Significant concentrations also occur along the Kanawha River in Putnam and Fayette Counties. These industrial areas have displaced agriculture and housing users, and new developments will continue this pattern into the future. One of the largest areas of potential industrial sites is in Mason and Putnam Counties on floodplain now used for agriculture. Floodplain along the Kanawha River is easily accessible to transportation systems, utilities and services. The large trained labor force, the relatively low cost of floodplain land, and the accessibility of raw materials has resulted in the present high degree of floodplain development.

3.2.2.12. **Existing Recreation**

Recreation is an authorized project purpose of the Bluestone Dam, and offers an abundance of quality experiences and diversity of opportunity in the area. Recreation was authorized from the Flood Control Act of 1944 that states “to construct, maintain, and operate public park facilities in reservoir areas”. The recreational opportunities include camping, picnicking, fishing, hunting, hiking, boating, sightseeing, wildlife viewing, kayaking, swimming, and other recreational pursuits. WRDA 1988 added the potential development of whitewater operations for enhancement of recreation. These seasonal whitewater operations were assessed for impacts on downstream water level fluctuations, downstream channel capacity, and maintenance of reservoir pool elevations for other project purposes. Because of the small difference in elevation between the summer pool EL 1,410-ft and winter pool EL 1,406-ft at Bluestone, the seasonal drawdown process is accomplished in just a few days. As a result, Bluestone whitewater releases are very rare and are for a short period of time.

3.2.3. **Future Without-Action Condition (FWAC)**

This section presents the assumptions and description of the FWAC (No Action) Plan. The FWAC forecast identifies the condition most likely to exist if the Federal government takes no action and considers what others (public, stakeholders, and the like) will do absent a Federal action. When the FWAC assumptions are considered in the quantitative FWAC risk assessment this becomes the FWAC (No Action) plan and the basis for which RMPs are evaluated and compared. In conclusion, the FWAC (No Action) Plan is estimated to slightly reduce risk, but not enough to meet TRGs. The FWAC (No Action) Plan will be presented in Chapter 5.

This section also summarizes the areas considered in the FWAC: Bluestone Dam Features; Operations and Maintenance; Hydrology (Climate Change); Population at Risk Projections; Evacuation Routes; Socioeconomics; Warning Systems and Warning Diffusion; Mobilization; Environmental Resources; Land Use; and Recreation.

3.2.3.1. **FWAC Bluestone Dam Features**

The FWAC assumes the dam does not significantly change over the period of analysis from the existing condition (with remaining 66+/- anchors installed).
3.2.3.2. FWAC Operation and Maintenance

Absent action under the Dam Safety program, it is reasonable to assume that USACE will continue to own and maintain Bluestone Dam as described in the ECRA.

3.2.3.2.1. FWAC Gate Operations

Currently, the process to operate the crest gates is to open each gate from the spillway piers. Since opening the gates fully would increase the likelihood of a dam failure due to the likelihood of bedrock scour this could put project personnel in an unacceptable life safety risk. This will be evaluated when developing RMPs.

3.2.3.2.2. FWAC Gate Reliability

The condition of the dam is likely to fluctuate throughout the study period, particularly the mechanical and electrical gate operating equipment, some of which is in excess of 65 years old. The BCRA Report highlighted the need for maintenance on this equipment with many items overdue for replacement or rehabilitation. Since the BCRA Report was completed, all of the crest gate hoisting wire ropes have been replaced as well as two of the right-angle gear reducers (gates 7 and 9) have been rehabilitated. The gear reducers were rehabilitated due to failure during regular periodic testing operations.

For the update to the ECRA and development of the FWAC, the operating machinery for the 21 crest gates was modeled for probabilities of unsatisfactory performance for three different scenarios. Unsatisfactory performance is defined as failure of a crest gate to open upon demand resulting in no discharge through that gate bay unless the reservoir rises above the gate resulting in flow overtop the gate. Details of the models and the results can be found in Appendix A.5 of the Bluestone Dam Risk Assessment Technical Summary Report (US Army Corps of Engineers, Bluestone Dam Risk Assessment Technical Summary Report, 2016).

The first scenario was for the gate machinery’s existing condition which includes the recent replacement of the wire rope. This scenario is considered to be the existing condition. The results of the model indicate the probability of unsatisfactory performance (PUP) of any single gate is approximately 9.5%. This would lead to a most likely scenario of one to three gates out of the 21 failing to open on demand, with a 32% probability of at least three, reducing the capacity of the spillway.

The second scenario was for the gate machinery as it exist today, but is projected into the future to year 2061. This scenario was included to show the likely condition in the future given no major maintenance or rehabilitation of the equipment. The results show that there is approximately 100% PUP of any single gate. This result leads to a most likely scenario of 19 out of the 21 gates failing to open on demand with a 17% probability of all 21 gates failing to open.

Due to this likely drastic decrease in reliability over the study period, a third scenario was included which assumed the gates were rehabilitated in 2016. It should be noted, that given the cost for a complete rehabilitation in excess of $10M, this did not occur in 2016 and is highly unlikely to occur within any budgeted year. The more likely scenario is to target lowest reliability components first, such as the case with the wire ropes in 2014, and complete the rehabilitation over several years within maintenance budget constraints. The failure probabilities for one rehabilitated gate operating machinery were then modeled into the future for reference as to how the numbers increase over that study period for the mechanical features only. The fault trees for the rehabilitated operating machinery were modeled as if the machinery was refurbished in 2016 and were maintained in the same condition over the next 25 years in increments of five years. In Figure 3-18 below, the PUP is approximately 0.5% for the 5 years following rehabilitation and increases to approximately 3% for the 25 years following rehabilitation.
The gate operating equipment consists of a large number of components but the primary driver for the reliability are the gear reducers which contribute approximately one-third of the PUP in the existing condition. Although the gate reliability will decrease over the study period even if they are rehabilitated, the PUP should remain sufficiently low such that the probability of multiple gates failing to open will remain very low.

The results indicate there would be a greater than 90% probability that no more than one gate should fail to open on demand through much, if not the full, study period given proper rehabilitation and maintenance of the gate machinery. The 2013 BCRA Report concluded that a single gate failing to open would have limited, to negligible, effect on the loading and response of the structure. Therefore, the FWAC assumes all gates operate properly for simplification.

3.2.3.2.3. Additional Maintenance Items

In addition to maintaining the gate operating equipment, additional maintenance items will be required to ensure the stability of the structure in the future. The two primary items are liftoff testing of the high capacity anchors and cleaning of the foundation drains. Monitoring and maintenance of the instrumentation will also be required in the future.

3.2.3.2.3.1. Liftoff Testing

Ideally liftoff testing of sentinel anchors (anchors designed to be restressable and easily accessible through manhole covers for the purpose of periodic evaluations) should be performed five years after the final anchor installation for each phase, then every ten years thereafter. This testing ensures that the anchors are performed as expected and the required load is available to resist sliding and overturning. The four anchors installed during the 2002 full scale anchor study have been liftoff tested twice, once shortly after installation to verify the lockoff load and approximately 6 months after lockoff to determine relaxation, and were confirmed to be performing adequately at that time. However, re-testing of the 2002 anchors and testing of Phase 2B anchors is currently overdue.

Because of the staggered construction phases and because of the expense with performing this type of work, the testing schedule will be modified to optimize the number of anchors that can be tested while adhering as closely as possible to the ideal schedule. Funding for liftoff testing of part of the nineteen Phase 2B anchors (completed in 2009) and for a second test of the four 2002 test anchors is available and the remainder has been requested for FY17. Phase 4 anchor installations should be completed in 2019 so liftoff testing should be performed no later than in 2024, but will likely be scheduled early in construction of RMP 6 to evaluate if the current design is performing adequately. All anchors will be liftoff tested at this time (four from the anchor study, nineteen from Phase 2B, and sixteen from Phase 4) which will allow them to be on the same testing schedule in the future. Approximately six anchors in the dam that are part of RMP 6 are planned to be sentinel anchors which will also require periodic liftoff testing. If installed early in the project, these anchors may be tested toward the end of construction. If so, all anchors can then be liftoff tested at 10-year intervals under the O&M Program. Costs for testing have been captured for construction of RMP 6 and for future O&M budgets.

3.2.3.2.3.2. Drain and Uplift Cell Piping Cleanout

Drain reaming and pressure washing of the uplift cell piping should continue to be performed at 10-year intervals. This work was last done in 2009 and will be required again in 2019 and every ten years thereafter. There are currently about 290 drains with a total of approximately 23,000 linear feet of drain to be reamed. There is approximately 4,575 linear feet of uplift cell piping to be pressure washed. Costs for drain reaming and pressure washing of uplift cell piping have been captured for future O&M budgets.
Figure 3-18: Probability of Unsatisfactory Performance of Rehabilitated Gate Machinery
3.2.3.2.4. FWAC Water Control Plan Assumptions

Once Phase 4 construction of the DSA project is complete, the threshold pool can be raised from EL 1,510-ft back to the original flood control pool of EL 1,520-ft. However, the restricted discharge of 140,000 cfs will likely remain as the threshold due to the weir instability. Although the risk assessment shows the probability of breach for a discharge of 140,000 cfs as fairly remote ($<1 \times 10^{-6}$), there is a lot of uncertainty on the exact nature of a potential breach of the weir and how failure may progress. Therefore, it is assumed the District would choose to open the penstocks prior to exceeding this threshold discharge through the primary spillway. This restriction of discharge through the primary spillway and addition of the penstocks as an auxiliary spillway will require an update to the water control plan. For details of the timeline of Water Control Plan Activities see Table 6-2 and Figure 6-15.

The FWAC risk assessment assumes the post Phase 4 DSA water control plan update is in place which will restrict flow due to the weir instability and allow operation to a flood control pool of EL 1,520-ft, and defines operation of the penstocks. If the reservoir is projected to exceed EL 1,520-ft, the spillway crest gates will be opened to increase discharge up the threshold of 140,000 cfs. This limit will be maintained until the reservoir reaches a pool EL 1,525-ft (approximate top of spillway crest gates when partially opened to restrict flow to 140,000 cfs). If the reservoir is projected to continue to rise, the penstocks would then be operated prior to fully opening the crest gates. The auxiliary penstock spillway was designed for emergency use to avoid overtopping, the high risk of breach from spillway flows exceeding levels that threaten structures within the stilling basin and leading to scour warrant earlier operation in the absence of modifications to the basin. In addition, once all penstocks are opened, the crest gates will be operated as necessary to prevent overtopping of the gates up to a fully opened condition. See Table 3-8.

<table>
<thead>
<tr>
<th>Table 3-8: FWAC Water Control Deviations</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Update to the Water Control Plan</strong></td>
</tr>
<tr>
<td><strong>FWAC Risk Assessment Assumptions for Water Control Plan</strong></td>
</tr>
</tbody>
</table>

An example deterministic routing using the 2003 flood hydrograph, scaled 7 times, with a starting pool of EL 1,410-ft and downstream flooding is illustrated in Figure 3-19 using the FWAC assumptions.
Figure 3-19: Example deterministic routing
3.2.3.3. FWAC Hydrology
Due to minimal anticipated future development upstream the existing and FWAC hydrology is the same for the DSMS. Additional details can be found in Appendix D.

3.2.3.4. FWAC Climate Preparedness and Resilience
There has been a substantial amount of study focused on evaluating the potential changes in hydrology associated with climate change. The majority of this study has been focused on the changes in trends associated with long term normal values (e.g. average annual precipitation or average annual discharge). There has not been equivalent research on quantifying changes of extremes of smaller spatial and temporal distribution; such as extreme flood hydrology and the estimates of the probable maximum flood. The hydrologic events associated with the driving potential failure mode and therefore the risk management plan described within this DSMS, are relatively low probability events that have no historic precedence; and therefore are an additional challenge in estimating changes due to long term climate variation.

In accordance with ECB No. 2016-25 - Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs, and Projects, a qualitative assessment of climate change impacts for Bluestone Dam watershed was completed. The objective of the assessment is to enhance USACE climate preparedness and resilience by incorporating relevant information about climate change impacts in hydrologic analyses. Additional details of this qualitative assessment can be found in Appendix D. Climate Change is also discussed in the SFEIS, Appendix K, in Section 4.7.2.3. The main conclusions of qualitative assessment are:

1. Variability of the climate within the region of the Bluestone Dam watershed has been identified, however, strong consensus of data was not detected.
2. Trend data from observed streamflow gage data within the Bluestone Dam watershed is not statistically significant.

USACE policy does not require a quantitative assessment of how climate change might impact PMF magnitudes for a study area. Since the relevant decisions for the DSMS are associated with extreme flood hydrology (i.e. PMF), this qualitative assessment will be used for record but will not influence decisions with respect to the DSMS.

3.2.3.5. FWAC Population at Risk Projections
In most of the communities surrounding Bluestone Reservoir, population change has been slow in recent decades (less than 1% per year). It is reasonable to assume that slow population changes will continue into the foreseeable future. The State of West Virginia provides projections through 2030. The projections are summarized in Table 3-9. The table also shows percent increase in the total population from 2010 to 2040. For the FWAC condition, the model results were factored by a weighted population index representing the year 2040. This year was selected based on the available population projection data in each state to be representative of regional expectations. Kentucky and Ohio both had established population projections out to 2040, but since West Virginia only has projections out to 2030 a trend line was used to extend the West Virginia population estimates to 2040. The FWAC population per structure determined from the Hydrologic Engineering Center, Flood Impact Analysis (HEC-FIA) software were indexed from 2010 to 2040 using a weighted average of 0.95 which was based on the maximum PAR and life loss breakouts by county. This overall decrease in population from the existing to FWAC is congruent with the declining population trend in the area as seen over the last several decades. See Table 3-10 for population at risk by scenario.
Table 3-9: Population Projections by County

<table>
<thead>
<tr>
<th>County</th>
<th>Max PAR</th>
<th>% of PAR</th>
<th>2010</th>
<th>2014</th>
<th>2020</th>
<th>2030</th>
<th>2040</th>
<th>Change 2014-2040</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cabell, WV</td>
<td>5,627</td>
<td>3.3%</td>
<td>96,319</td>
<td>97,109</td>
<td>98,679</td>
<td>99,753</td>
<td>101,704</td>
<td>5.6%</td>
</tr>
<tr>
<td>Fayette, WV</td>
<td>4,451</td>
<td>2.6%</td>
<td>46,039</td>
<td>45,132</td>
<td>44,611</td>
<td>42,795</td>
<td>41,295</td>
<td>-10.3%</td>
</tr>
<tr>
<td>Jackson, WV</td>
<td>264</td>
<td>0.2%</td>
<td>29,211</td>
<td>29,126</td>
<td>28,904</td>
<td>28,305</td>
<td>27,892</td>
<td>-4.5%</td>
</tr>
<tr>
<td>Kanawha, WV</td>
<td>117,407</td>
<td>68.1%</td>
<td>193,063</td>
<td>190,223</td>
<td>189,173</td>
<td>182,156</td>
<td>177,432</td>
<td>-8.1%</td>
</tr>
<tr>
<td>Lincoln, WV</td>
<td>201</td>
<td>0.1%</td>
<td>21,720</td>
<td>21,561</td>
<td>21,054</td>
<td>19,607</td>
<td>18,657</td>
<td>-14.1%</td>
</tr>
<tr>
<td>Mason, WV</td>
<td>2,229</td>
<td>1.3%</td>
<td>27,324</td>
<td>27,016</td>
<td>27,298</td>
<td>27,029</td>
<td>26,973</td>
<td>-1.3%</td>
</tr>
<tr>
<td>Nicholas, WV</td>
<td>117</td>
<td>0.1%</td>
<td>26,233</td>
<td>25,827</td>
<td>25,878</td>
<td>24,485</td>
<td>23,824</td>
<td>-9.2%</td>
</tr>
<tr>
<td>Putnam, WV</td>
<td>13,760</td>
<td>8.0%</td>
<td>55,486</td>
<td>56,770</td>
<td>57,668</td>
<td>57,726</td>
<td>59,115</td>
<td>6.5%</td>
</tr>
<tr>
<td>Raleigh, WV</td>
<td>90</td>
<td>0.1%</td>
<td>78,859</td>
<td>78,241</td>
<td>78,028</td>
<td>75,813</td>
<td>74,563</td>
<td>-5.4%</td>
</tr>
<tr>
<td>Summers, WV</td>
<td>949</td>
<td>0.5%</td>
<td>13,927</td>
<td>13,417</td>
<td>13,756</td>
<td>13,256</td>
<td>13,051</td>
<td>-6.3%</td>
</tr>
<tr>
<td>Wayne, WV</td>
<td>554</td>
<td>0.3%</td>
<td>42,481</td>
<td>41,122</td>
<td>40,461</td>
<td>38,324</td>
<td>36,361</td>
<td>-14.4%</td>
</tr>
<tr>
<td>Gallia, OH</td>
<td>8,160</td>
<td>4.7%</td>
<td>30,934</td>
<td>30,397</td>
<td>30,600</td>
<td>30,250</td>
<td>30,280</td>
<td>-2.1%</td>
</tr>
<tr>
<td>Lawrence, OH</td>
<td>9,830</td>
<td>5.7%</td>
<td>62,450</td>
<td>61,623</td>
<td>62,390</td>
<td>62,390</td>
<td>62,680</td>
<td>0.4%</td>
</tr>
<tr>
<td>Meigs, OH</td>
<td>1,665</td>
<td>1.0%</td>
<td>23,770</td>
<td>23,331</td>
<td>23,630</td>
<td>23,170</td>
<td>22,340</td>
<td>-6.0%</td>
</tr>
<tr>
<td>Scioto, OH</td>
<td>1,341</td>
<td>0.8%</td>
<td>79,499</td>
<td>77,258</td>
<td>77,430</td>
<td>75,520</td>
<td>77,660</td>
<td>-2.3%</td>
</tr>
<tr>
<td>Boyd, KY</td>
<td>1,288</td>
<td>0.7%</td>
<td>49,542</td>
<td>48,832</td>
<td>49,446</td>
<td>48,378</td>
<td>46,843</td>
<td>-5.4%</td>
</tr>
<tr>
<td>Greenup, KY</td>
<td>4,581</td>
<td>2.7%</td>
<td>36,910</td>
<td>36,308</td>
<td>36,923</td>
<td>36,231</td>
<td>35,008</td>
<td>-5.2%</td>
</tr>
</tbody>
</table>

*Max PAR represents the IDF event Daytime PAR using 2014 estimates.*

Table 3-10: Population at Risk (PAR) by Scenario

<table>
<thead>
<tr>
<th>HEC-FIA Scenario</th>
<th>Breach</th>
<th>Non-Breach</th>
<th>Incremental</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Daytime PAR</td>
<td>Nighttime PAR</td>
<td>Daytime PAR</td>
</tr>
<tr>
<td>1552 Breach – 22 Monoliths</td>
<td>165,302</td>
<td>123,440</td>
<td>155,186</td>
</tr>
<tr>
<td>1535 Breach – 22 Monoliths</td>
<td>152,607</td>
<td>111,179</td>
<td>138,295</td>
</tr>
<tr>
<td>1552 Breach – Abutment</td>
<td>164,179</td>
<td>122,044</td>
<td>155,186</td>
</tr>
<tr>
<td>1542 Breach – Abutment</td>
<td>156,041</td>
<td>114,559</td>
<td>145,655</td>
</tr>
<tr>
<td>1538 Breach – Abutment</td>
<td>153,254</td>
<td>112,065</td>
<td>141,876</td>
</tr>
<tr>
<td>1552 Breach – 11 Monoliths</td>
<td>163,886</td>
<td>121,710</td>
<td>155,186</td>
</tr>
<tr>
<td>1542 Breach – 11 Monoliths</td>
<td>155,877</td>
<td>114,359</td>
<td>145,655</td>
</tr>
<tr>
<td>1538 Breach – 11 Monoliths</td>
<td>152,933</td>
<td>111,726</td>
<td>141,876</td>
</tr>
<tr>
<td>1535 Breach – 11 Monoliths</td>
<td>150,391</td>
<td>109,075</td>
<td>138,295</td>
</tr>
<tr>
<td>1529 Breach – 11 Monoliths</td>
<td>135,419</td>
<td>95,196</td>
<td>123,751</td>
</tr>
<tr>
<td>1524 Breach – 11 Monoliths</td>
<td>122,339</td>
<td>83,599</td>
<td>107,618</td>
</tr>
<tr>
<td>1519 Gates Half Open Breach – 11 Monoliths</td>
<td>111,109</td>
<td>72,084</td>
<td>21,177</td>
</tr>
<tr>
<td>1519 Gates Closed Breach – 11 Monoliths</td>
<td>109,900</td>
<td>70,741</td>
<td>604</td>
</tr>
<tr>
<td>1433 Breach – 11 Monoliths</td>
<td>1,155</td>
<td>1,449</td>
<td>No damaging releases</td>
</tr>
<tr>
<td>1410 Breach – 11 Monoliths</td>
<td>400</td>
<td>488</td>
<td></td>
</tr>
</tbody>
</table>

PAR – FWAC

81 June 2017
3.2.3.6. FWAC Evacuation Routes
The FWAC assumes that there will be minimal changes from the evacuation routes modeled in the existing condition risk assessment. The assumption of minimal changes over the planning horizon is due to the topography as a limiting factor and no new major highways are expected to be built.

3.2.3.7. FWAC Socioeconomics
The general economic characteristics of the study area are not expected to change significantly in the foreseeable future. This is unlikely to change dramatically over the planning horizon. While there are changes to the economy resulting from a decline in the coal industry, increases in tourism and the service sector are expected to result in a minimal overall change. Population projections in the study area indicate changes (both gains and losses) throughout the study area. However, minimal changes are expected overall. It is reasonable to assume that the lake will still be widely used as recreational resource.

3.2.3.8. FWAC Warning Systems and Warning Diffusion
In the 2013 BCRA, the HEC-FIA modeling used to determine life safety risk was highly sensitive to changes in the estimated mobilization of the population in and around Charleston, West Virginia. Since there has been no significant flooding in the Kanawha Valley since the early 20th century, there is significant uncertainty regarding the effectiveness of mobilization. To address this uncertainty and develop a more confident estimate, LRH held an Expert Opinion Elicitation (EOE) in January of 2014 involving a small group of expert representatives from state and local emergency management agencies. The purpose of this EOE was to identify the current and future state of evacuation planning and estimate potential mobilization scenarios. Background information on existing planning efforts, geographic and social conditions, and emergency preparedness and response capabilities was gathered and documented. The outcome of the EOE included an estimate of mobilization given existing conditions and capability, an estimate of mobilization given the most likely future conditions and capabilities, and an optimistic estimate to represent uncertainty in future capabilities and create a range around the most likely estimate. A detailed report documenting the EOE is included in Appendix J.

The warning diffusion curves used in HEC-FIA modeling were based in part on methods established by USACE incorporating research by Mileti & Sorensen. The method involves the use of an interview schedule with a series of questions to evaluate what type of emergency response planning and capabilities are in place, how warning and evacuation messages would be crafted, what type of messages and content would be released, and a multitude of other factors. The evaluation creates a quantified score used in an equation-based model developed by Mileti & Sorenson to estimate probable warning diffusion and mobilization curves. The evaluation for this current consequence analysis was completed based on information obtained during the January 2014 EOE.

The interview schedule questions were answered based on current conditions to create an existing condition curve, and then some of those answers were modified to create a FWAC mobilization curve representing likely future improvements that will result in better mobilization. The interview schedule and Mileti & Sorenson equations were used to create the shapes of the existing and FWAC curves.

The primary difference in the warning diffusion curve between the existing and FWAC is based on an assumption of general across the board improvements in warning systems and capabilities. This assumption of improved warning systems was not necessarily based on specific improvements in the short term such as the addition of more flood warning sirens; rather, it was based on the estimated broad improvements in communication technology over the next 20-30 years. The team felt that changes caused by broad technological improvements would have a significantly greater impact on warning capabilities than any specific improvements or additions such as additional sirens. The FWAC is a 50 year period from
2020 to 2070, the midpoint being the year 2045, so the warning curve improvement from existing to FWAC is meant to represent the potential improvements in all warning systems between the current time and 2045 in order to represent an average warning system characterization for the entire FWAC period.

As of April 2017, the City of Hinton, community at most risk, has made progress by purchasing and installing a flood warning siren, but it hasn’t been connected or tested. The West Virginia Division of Military Affairs and Public Safety continues to make progress on updating emergency response planning and seeking grants to install more advanced flood warning systems for downstream local emergency planning commissions and first responders. The focus so far has been on the City of Charleston, one of the major consequence centers. They have an existing evacuation plan, but is several years out of date. The City of Charleston’s evacuation plan can be found at http://kanawha.us/documents/Kanawha-Charleston_Evacuation_Plan.pdf. As discussed elsewhere, voice capability sirens are also being installed with Charleston.

Additionally a significant flood event occurred in June 2016, in which up to 10 inches of rain fell in parts of West Virginia. Some of the areas impacted were downstream of three flood risk management projects, Summersville Dam, Sutton Dam and Bluestone Dam. There were significant consequences including 23 lost lives and these have highlighted the need to continue to make progress on this planning.

It is important to note that changes to the warning diffusion curve that represents how fast a warning will be distributed have little to no impact on the estimated life loss for any scenarios involving spillway flow. This is because flood and evacuation warnings would be based relative to that spillway flow, which would be forecasted in advance by rainfall gauges and other observations. The spillway flow would cause very significant flooding and require the warning and evacuation of the same areas that would be impacted by a breach. Since the majority of the risk at the project involves scenarios with spillway flow, the life safety risk is not impacted by changes to how fast that warning would be distributed; in nearly all cases with spillway flow population would receive a warning prior to the arrival of flood waters. Because the warning diffusion curve has no impact on life safety risk, there was no reason to go beyond the assumption of general FWAC improvements into attempts at estimating how specific improvements in warning systems could impact the warning diffusion.

3.2.3.9. FWAC Mobilization

Similar to the warning diffusion curve, the mobilization curve was also based in part on methods established by USACE incorporating research by Mileti & Sorensen. The mobilization curve represents how many people over time will actually mobilize and evacuate once they have received a warning. Once people receive a warning they typically do not immediately evacuate; they may delay to gather more information, gather belongings, re-unite with family members, etc. Even then, some people may be unwilling or physically unable to evacuate. For this reason, a mobilization curve typically does not reach 100%.

The primary driver of life safety consequences of Bluestone Dam is the mobilization curve, and the primary difference between the existing and FWAC consequences is the change in that curve. Coordination with representatives from state government officials, local government officials and emergency management officials within the downstream communities indicates that risk preparedness will likely be improved within the period of analysis through the implementation of enhanced warning systems and evacuation plans. In the future conditions, such improvements will mean warnings can go out over more channels and contain better information. It has been shown through research that warning message content and multi-channel distribution has a significant effect on whether or not people choose to evacuate and how they accomplish that evacuation. The improvements will also combine with gradual improvements in
emergency planning, both at the local and regional level and in the community of practice as a whole. These changes should result in a mobilization curve that increases at a faster rate in the FWAC as opposed to the existing condition and concludes at a higher overall mobilization percentage.

The Mileti & Sorenson interview schedule questions were answered based on current conditions to create an existing condition curve, and then some of those answers were modified to create a FWAC mobilization curve representing likely future improvements that will result in better mobilization. The interview schedule and Mileti & Sorenson equation were used to create the shapes of the existing and FWAC curves, but the equation based curves must be further modified by estimating maximum mobilization rates for each curve. This results in a Mileti & Sorenson shaped curve that caps out at a maximum rate at the 8-hour time point. A further 24-hour maximum rate was also estimated, meaning that mobilization percent slowly increases in a linear fashion between 8 hours and 24 hours. Different maximum 24-hour rates were assumed for impact areas along the New and Kanawha Rivers as opposed to those along the Ohio River with the assumption that much longer warning times, better experience and perception of flooding, and more opportunity for overall preparedness will result in higher mobilization potential along the Ohio River. In summary, there are two 8-hour curves, one each for the existing and FWAC scenarios, and each curve is further modified by the addition of a 24-hour point where mobilization is dependent upon whether the area is along the New/Kanawha Rivers or along the Ohio River. Each curve and maximum mobilization percent also includes minimum and maximum estimates for a triangular distribution of uncertainty.

<table>
<thead>
<tr>
<th>NEW/KANAWHA RIVER COUNTRIES</th>
<th>OHIO RIVER COUNTRIES</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Existing Max Mobilization Percent</strong></td>
<td><strong>Existing Max Mobilization Percent</strong></td>
</tr>
<tr>
<td>Time</td>
<td>Minimum</td>
</tr>
<tr>
<td>8 Hours</td>
<td>80</td>
</tr>
<tr>
<td>24 Hours</td>
<td>85</td>
</tr>
</tbody>
</table>

| **FWAC Max Mobilization Percent** | **FWAC Max Mobilization Percent** |
| Time | Minimum | Most Likely | Maximum | Time | Minimum | Most Likely | Maximum |
| 8 Hours | 88 | 93 | 99 | 8 Hours | 90 | 93 | 99 |
| 24 Hours | 90 | 96 | 99.5 | 24 Hours | 95 | 97.5 | 99.5 |

Figure 3-20 and Figure 3-21 represent the first 8 hours of the mobilization curves based on the Mileti & Sorenson method for the ECRA and FWAC.
Figure 3-20: Existing Condition Mobilization curve, first 8 hours

Figure 3-21: FWAC Mobilization curve, first 8 hours
During evaluation of warning and mobilization potential, it is important to understand the timing of the breach and non-breach events. The figure below represents flow hydrographs which depict the inflow at the dam, the outflow at the dam, and the flow at Charleston, West Virginia for two different scenarios; the EL 1,542-ft breach (in gray/black) and the EL 1,524-ft breach (in red). At the EL 1,542-ft event, spillway flow causes flood stage to be exceeding in Charleston after about 10 hours, and that spillway flow can be projected earlier based on rainfall gage data which could provide an additional four or more hours of warning. There is less warning opportunity time in the EL 1,524-ft event because the breach would occur only about three hours after the spillway flow begins. However, in most cases with spillway flow there will be enough time for the warning and mobilization curves to reach their upper boundaries, with results for life loss being highly dependent on what the mobilization upper boundaries are estimated to be.

**Figure 3-22: Flow Hydrographs at Bluestone Dam and Charleston, West Virginia**
3.2.3.10. FWAC Environmental Resources

The Huntington District assessed data pertinent to coal mining, water quality, endangered species, industrial development downstream, and tourism. Initial assessments indicate little change is expected to existing environmental conditions during the 50-year period of analysis. LRH has coordinated this forecast with the West Virginia Division of Natural Resources and the US Fish and Wildlife Service. Much of what was detailed in the 1998 DSA Report remains sufficient to describe existing environmental conditions we see today in the New and Kanawha valleys. Without the forecast of any large landscape changes in the study area, it is reasonable to conclude that the FWAC, with respect to environmental conditions, will remain consistent with the existing condition. A summary of this assessment is included in Appendix J.

3.2.3.11. FWAC Land Use

Generally speaking, floodplain areas of the Kanawha River in Kanawha and Putnam counties (with some exceptions for larger corporately-owned tracts destined for industrial development) are built out and any changes would be replacement of existing development with similar land uses due to current land use regulations (zoning) and adjacent landowner pressures. Future changes in municipal zoning ordinances recognizing the risks presented by breach and non-breach flows from Bluestone Dam could minimize increases in potential loss of life (restrict redevelopment to commercial or recreational land uses or lower density uses) but would have little effect on economic losses associated with land uses that would likely replace residential uses under these controls. However, due to the topography there is limited developable land that is not already built out; therefore, the potential for zoning to impact life loss is minimal.

Several large, corporately-owned, vacant tracts exist in Kanawha and Putnam counties adjacent to the river that may be developable in the future, but it is likely that their location adjacent to lower-cost river transportation and high market values would point towards an industrial or commercial usage rather than residential uses. Although the long-term future (50 years) of current at-risk major industries in the Kanawha River Valley (i.e. chemical industry) is highly uncertain, future changes in corporate strategy, shifts in international trading and production, more restrictive environmental regulations, and aging infrastructure replacement costs could result in a reduction of at-risk facilities in the future. However, without extensive remediation the Hazardous Toxic Radioactive Waste (HTRW) footprint present at those sites would remain into the future.

The remaining uncertainty for future development downstream from Bluestone Dam is Mason County at the farthest downstream location on the Kanawha River. Coordination with the current Director of the Region 2 Planning and Development area (which includes Mason County) and the past Director of the Putnam County Development Authority provided detailed information on the past, present and future development of this lower Kanawha River and Ohio River land area. Starting at the downstream boundary of Buffalo, West Virginia (approximately river mile 21.5), and ending at the upstream incorporated limits of Henderson and Point Pleasant, West Virginia there are hundreds of acres of developable land on both sides of the river now dedicated to agricultural uses and scattered farm related residential uses. In comparison to the densities of residential commercial and industrial development in Putnam County located immediately upstream, Mason County floodplains on the Kanawha River are largely vacant.

The primary reason for the lack of past development is the ownership of that property. Much of that vacant land is owned by one corporate owner that leases tracts of land for agricultural uses and other non-development purposes, but has never shown interest in subdivision of the property for residential and/or commercial uses. Some smaller tracts of land within this large holding have been sold for critically
needed public uses, but the corporate owner maintains these tracts of land for future development that satisfies its own future corporate objectives and/or for industrial users that would take advantage of the economics of cheaper water transportation on the Kanawha River, the reliable source of fresh water for processing and cooling provided by the RC Byrd navigation pool, comparatively inexpensive energy rates provided by the current landowner for industrial users, and relatively low land development costs afforded by the flat floodplain topography.

This same corporate strategy applies to several hundred acres of land along the Ohio River both upstream and downstream of the Kanawha River juncture in Ohio and West Virginia (same corporate owner) resulting in limited future development by individual residential and commercial uses.

Absent future changes in this corporate strategy brought about by significant changes in the energy industry or other outside influences, future development of this property will not likely add to the loss of life potential already identified in Summers, Fayette, Raleigh, Kanawha and Putnam counties, but future development for industrial land uses (part of the current corporate strategy) may add to potential economic impacts due to breach or non-breach flows from Bluestone Dam.

3.2.3.12. FWAC Recreation
Initial assessments indicate little change is expected to existing recreational resource conditions during the 50-year period of analysis. The types of recreational opportunities on the upstream and downstream of the dam would remain similar to the existing condition.
4. ECRA & FWAC RISK ASSESSMENTS
The risk management framework is a process that identifies Potential Failure Modes (PFM) that have associated risks that are above the agencies tolerable risk guidelines; and therefore require implementation of risk controls to reduce risk of dam failure. These risk controls, or RMPs in the case of the DSMS, are formulated to meet the study specific objectives that are largely based on the probability and consequences of a dam failure for both existing and projected future without action conditions (FWAC). Figure 4-1 illustrates the dam safety risk framework.

Figure 4-1: Dam Safety Risk Framework (US Army Corps of Engineers, Engineering Regulation (ER) 1110-2-1156, Safety of Dams - Policy and Procedures, 2014)

4.1. Previous Risk Assessments
Several risk assessments were completed for Bluestone Dam prior to the Existing Condition Risk Assessment (ECRA) and Future without Federal Action Condition (FWAC) Risk Assessment. Figure 4-2 shows an historical timeline of risk assessments completed for Bluestone Dam. The following sections summarize each risk assessment and their conclusions.

4.1.1. Screening Portfolio Risk Assessment
In June 2005, a Screening Portfolio Risk Assessment (SPRA) was completed. The PFMs of highest concern were foundation stability of the dam and abutments and structural stability of the spillway stilling basin.
The SPRA assessment took place prior to installation of monolith anchors and construction of the auxiliary penstock stilling basin. Based on SPRA findings, the Dam Senior Oversight Group (DSOG) rated Bluestone Dam as a DSAC 2 in December 2008.

4.1.2. USBR / USACE Risk Assessment
A Potential Failure Mode Analysis (PFMA) and abbreviated risk assessment was conducted in 2008. All the previous assessments and analysis completed prior to 2008 were done using a deterministic analysis. The USACE began moving towards using risk informed decision making around this time and a risk assessment team comprised of members of the U.S. Bureau of Reclamation (USBR) and the USACE Risk and Reliability Directorate of Expertise Cadre performed a Potential Failure Mode Analysis (PFMA) and prepared a qualitative risk assessment. The findings of this effort identified five potential dam safety concerns:

- Scour,
- Factors of safety,
- Rock strength,
- Loss of life, and
- Flood frequency curves (superseded by the 2013 Baseline Condition Risk Assessment)

This assessment concluded risk following the completion of the DSA project remained unacceptable and expedited action was needed. As a result, a workshop was held in October 2008 in order to reach a resolution for addressing the identified issues of concern and failure modes. While ongoing design and construction efforts of features approved under the DSA Evaluation Report were underway, the District was instructed to prepare a DSMR to supplement the 1998 DSA Report to reduce risk associated with the failure modes listed above. These failure modes are predominately related to the spillway component of the dam.

4.1.3. Potential Failure Mode Analysis (PFMA)
A formal facilitated PFMA was completed in October 2012. Thirty-five potential failure modes were identified. A list of the PFMs can be found in Table 4-2 along with their conclusions from each completed risk assessment.

4.1.4. Baseline Condition Risk Assessment
The BCRA was completed in July 2013 which quantified incremental risk associated with dam failure. This risk assessment was conducted with the assumption that construction of phase 3 and 4 of the DSA project, as well as installation of the remaining dam monolith anchors was in place. There are some features which were approved in the DSA study which have not been constructed and were not considered to be in place for this risk assessment. These features include things such as the parapet wall, apron anchors and vertical training wall extension. The features not considered as part of the BCRA mostly deal with the stilling basin and reducing overtopping. Stilling basin scour and potential for overtopping were the main dam safety concerns identified during the 2008 USBR/USACE risk assessment.

The BCRA reevaluated the PFMs that were identified in the PFMA and determined that many of them were not risk drivers as standalone PFMs; however, when combined they became risk drivers. The BCRA concluded that PFM 33, 34 and 35 were the primary risk drivers at Bluestone Dam. PFM 33 – spillway monolith instability, is a combination of multiple PFMs identified in the PFMA and has the highest risk associated with dam failure. PFMs 34 and 35 are also considered risk drivers and are a combination of multiple failure modes identified during the PFMA. PFM 34 is non-overflow monolith instability and PFM 35 is abutment monolith instability. Details of these failure modes can be found in Section 4.3.
The BCRA was presented to the DSOG in July 2013 and the DSAC 2 was reaffirmed by the USACE Dam Safety Officer in a memorandum dated 18 September 2013 (Martin, PE, PMP, 2017).

### 4.2. Previous Studies/Investigations

The investigations and studies that influence the understanding of the risk associated with Bluestone Dam are presented in Table 4-1. An explanation is also provided on how each investigation improved the quality and/or reduced the uncertainty associated with the existing and future without Federal action risk estimates and the with Federal action risk-reduction estimates. Additional details can be found in the corresponding appendix as state in the last column of the table.

#### Table 4-1: Summary of Previous Studies

<table>
<thead>
<tr>
<th>Investigation</th>
<th>Description</th>
<th>Influence on Risk Understanding</th>
<th>Corresponding Appendix</th>
</tr>
</thead>
<tbody>
<tr>
<td>1974-1975: Foundation assessment and subsurface explorations and laboratory testing</td>
<td>Review of existing foundation information, drilling of borings in the gallery inspection holes and in the vicinity of the penstock and non-overflow sections, laboratory testing for rock strength parameter selection, sounding of foundation drains</td>
<td>Identification of potential failure mode (fault) not evaluated in original design, support updated stability analysis with new/assumed shear strengths to assess if stability issue existed</td>
<td>E</td>
</tr>
<tr>
<td>1984-1985: Subsurface explorations and laboratory testing</td>
<td>Drilling in gallery, laboratory testing for selection of rock strengths to confirm previous assumptions</td>
<td>Reduced uncertainty associated with foundation conditions in four monoliths across dam, reduced uncertainty in rock strengths used for stability analysis</td>
<td>E</td>
</tr>
<tr>
<td>1994: Subsurface explorations and additional foundation drainage</td>
<td>Drilling information used to correlate geology along dam axis, borings kept as drains and uplift cells</td>
<td>Reduced uncertainty associated with foundation conditions in valley bottom, reduced risk through improved uplift relief</td>
<td>E</td>
</tr>
<tr>
<td>1999-2001: Subsurface explorations and laboratory testing</td>
<td>Drilling and testing program for abutments and valley bottom, and collection of other field data such as i-angle measurements and pressure testing, soil borings and sampling on left abutment</td>
<td>Reduced uncertainty associated with foundation conditions, selection of rock strength parameters, support stability analysis and appropriate design of various features including anchors and possible thrust blocks for abutments</td>
<td>E</td>
</tr>
<tr>
<td>2008: Subsurface explorations, field i-angle measurements, laboratory testing</td>
<td>Drilling and testing program for updated shear strengths for valley bottom and collection of i-angle measurements and pressure testing</td>
<td>Reduced uncertainty associated with potential failure planes in the foundation, selection of rock strength parameters</td>
<td>E</td>
</tr>
<tr>
<td>2009, 2010: Subsurface explorations</td>
<td>Drilling for correlation of stratigraphy and fault to support Phase 3 design</td>
<td>Reduced uncertainty associated with potential failure planes in the foundation, characterization of rock erodibility, appropriate design of various features such as foundations and anchors</td>
<td>E</td>
</tr>
<tr>
<td>2011: Subsurface explorations</td>
<td>Correlate stratigraphy in the main stilling basin to identify foundation</td>
<td>Reduced uncertainty associated with potential failure planes in the</td>
<td>E</td>
</tr>
</tbody>
</table>
## Previous Investigations

<table>
<thead>
<tr>
<th>Investigation</th>
<th>Description</th>
<th>Influence on Risk Understanding</th>
<th>Corresponding Appendix</th>
</tr>
</thead>
<tbody>
<tr>
<td>2012: Subsurface explorations and laboratory testing</td>
<td>Drilling in left training wall and foundation, testing of concrete lift joint samples</td>
<td>Reduced uncertainty of selected shear strength for internal stability analysis of dam, correlate geology</td>
<td>E</td>
</tr>
<tr>
<td>2013: Subsurface explorations</td>
<td>Drilling downstream of second stage to define geology for design of DSMS alternatives</td>
<td>Reduced uncertainty associated with potential failure planes in the foundation, support characterization of rock erodibility, design of various features such as foundations and anchors</td>
<td>E</td>
</tr>
<tr>
<td>1998 DSA</td>
<td>Dam Safety Assurance study conducted based on a 1980 analysis of the PMF that identified a hydrologic deficiency</td>
<td>Lead to multi-phase multi-year investment 2000-ongoing). Conclusion of these investments were baselined for estimating existing and future risks.</td>
<td>A</td>
</tr>
<tr>
<td>2002 Anchor Test Program</td>
<td>Installation and testing to failure of eight 18-strand test anchors in the abutment and installation of four 61-strand production anchors in the dam</td>
<td>Reduced uncertainty associated with selection of bond strengths, reduced construction and cost risk through identification and resolution of constructability issues</td>
<td>E</td>
</tr>
<tr>
<td>2005 SPRA</td>
<td>Concerns with stability of the dam and abutments and structural stability of the spillway stilling basin</td>
<td>Rated DSAC 2</td>
<td></td>
</tr>
<tr>
<td>2008 PFMA and abbreviated risk assessment</td>
<td>USBR &amp; USACE conducted a Risk-Informed Issue Evaluation Study to evaluate the investment begin implemented by the DSA project</td>
<td>Identified potential failure modes not be addressed by the DSA project and supporting prioritization of implementation of the DSA project.</td>
<td>A</td>
</tr>
<tr>
<td>2012 Preliminary Hydraulic Modeling of Initial Array of Potential Stilling Basin Modifications</td>
<td>Stilling basin alternatives with a movable bed were evaluated to analyze hydraulic performance</td>
<td>Determined initial viability of Remote Conventional up to previous IDF (EL. 1,542-ft).</td>
<td>D</td>
</tr>
<tr>
<td>2013 BCRA</td>
<td>Completed to finalize the preliminary conclusions from the 2008 IES</td>
<td>Established actionable potential failure modes that were not addressed by the DSA project and directed the initiation of a DSMS. Confirmed DSAC 2</td>
<td>A</td>
</tr>
<tr>
<td>2013-15: PMP and PMF Study</td>
<td>Updated the hydrology based on the site-specific PMP to revise the Inflow Design Flood</td>
<td>Established an upper bound for hydrologic loading.</td>
<td>D</td>
</tr>
<tr>
<td>2014 Preliminary Hydraulic Modeling for Super-cavitating Baffle and Transitional Flip Basin Alternatives</td>
<td>Stilling basin alternatives were evaluated to analyze hydraulic performance for the revised Inflow Design Flood</td>
<td>Analyzed the risk reduction effectiveness of the RMPs.</td>
<td>D</td>
</tr>
<tr>
<td>2015 Penstock Stilling Basin Scour</td>
<td>Operational scenarios were modeled to identify sequences of adequate performance and to estimate scour potential downstream of the basin</td>
<td>Mitigated risk by identifying adequate operational sequences to minimize scour potential downstream of the basin and facilitated the design of the basin scour features</td>
<td>D</td>
</tr>
</tbody>
</table>
### Previous Investigations

<table>
<thead>
<tr>
<th>Investigation</th>
<th>Description</th>
<th>Influence on Risk Understanding</th>
<th>Corresponding Appendix</th>
</tr>
</thead>
<tbody>
<tr>
<td>2016 ECRA &amp; FWAC</td>
<td>Updated the 2013 BCRA with the new PMP and PMF results and also projected the incremental risks Bluestone Dam into a future where no further dam safety modification are made beyond Phase 4 of the DSA</td>
<td>Refined actionable failure modes to PFM 33 – Spillway Monolith Instability and confirmed that dam safety risks are currently and will remain above USACE tolerable risk guidelines in a future where no further dam safety modifications are made. Confirmed DSAC 2</td>
<td>A</td>
</tr>
<tr>
<td>2016 Phase A Modeling</td>
<td>RMP 6 was modeled to develop hydraulic loads and estimated scour potential downstream of the primary basin</td>
<td>Provided additional information on the effectiveness of the selected plan, supported hydraulic loading estimates, and evaluated the need for specific RMP 6 components.</td>
<td>D &amp; E</td>
</tr>
<tr>
<td>1996-2016 Environmental Studies Environmental Assessments</td>
<td>Two Environmental Impact Statements and several Environmental Assessments have been conducted to support Dam Safety actions. These are focused on defining the affected environment, socioeconomics, recreation, terrestrial and aquatic habitat, cultural resources and analyzing impacts for significance determinations</td>
<td>Defined, per NEPA and applicable resource laws and regulations what impacts would occur from implementation of dam safety actions as well as defined what measures were appropriate to avoid, minimize or otherwise mitigate for any significant impacts</td>
<td>K</td>
</tr>
</tbody>
</table>

#### 4.3. ECRA/FWAC Overview

This report presents the results of the final risk assessment for the existing condition and future without Federal action condition risk assessments. There were four potential failure modes that were analyzed as part of the risk assessment. Potential failure mode (PFM) 33, spillway monolith instability, showed to be the only potential failure mode that exceeded tolerable risk guidelines. The report concluded that the risk associated with the remaining two risk–driving PFMs (PFM 34 - Non-overflow Monolith Instability, and PFM 35- Abutment Monolith Instability) considered in the 2013 BCRA, were within tolerable risk guidelines. However, taking into account uncertainty, the team decided to retain PFM 34 and PFM 35 for risk reduction opportunities if found to be economically feasible. The fourth PFM analyzed was penstock monolith instability (PFM 9), in which a semi-quantitative assessment was performed resulting in a likely remote conclusion. There was uncertainty associated with this likely remote failure mode. This uncertainty was later reduced through physical hydraulic modeling and confirmed as a remote, or non-risk driving, PFM. Due to this failure mode’s remote likelihood, it is not considered a dam safety issue and risk reduction measures were not formulated to address this PFM.
Table 4-2: History of Potential Failure Modes at Bluestone Dam

<table>
<thead>
<tr>
<th>PFM</th>
<th>Failure Mode Description</th>
<th>2012 PFMA</th>
<th>2013 BCRA</th>
<th>2016 ECRA</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Penstock Gate Failure</td>
<td>Insignificant</td>
<td>Not carried forward</td>
<td>Not carried forward</td>
</tr>
<tr>
<td>2A</td>
<td>Overtopping of Abutment Monoliths leads to scour and breach due to sliding</td>
<td>Potentially Significant</td>
<td>Part of PFM 35</td>
<td>Part of PFM 35</td>
</tr>
<tr>
<td>2B</td>
<td>Overtopping of Non-Overflow (or Assembly bay) Monoliths leads to scour and breach due to sliding</td>
<td>Potentially Significant</td>
<td>Part of PFM 34</td>
<td>Part of PFM 34</td>
</tr>
<tr>
<td>2C</td>
<td>Overtopping of Penstock Monoliths Leading to Monolith Instability</td>
<td>Insignificant</td>
<td>Not carried forward</td>
<td>Not carried forward</td>
</tr>
<tr>
<td>3</td>
<td>Overtopping floods the galleries and causes an increase in uplift leading to breach due to sliding</td>
<td>Potentially Significant</td>
<td>Part of PFM 33, 34, &amp; 35</td>
<td>Part of PFM 33, 34, &amp; 35</td>
</tr>
<tr>
<td>4</td>
<td>Spillway Flow causes scour leading to breach due to sliding</td>
<td>Potentially Significant</td>
<td>Part of PFM 33</td>
<td>Part of PFM 33</td>
</tr>
<tr>
<td>5</td>
<td>Structural failure of crest gates</td>
<td>Insignificant</td>
<td>Not carried forward</td>
<td>Not carried forward</td>
</tr>
<tr>
<td>6</td>
<td>Failure to open one or more crest gates leads to overtopping (mechanical equipment failure)</td>
<td>Potentially Significant</td>
<td>Part of PFM 33, 34, &amp; 35</td>
<td>Part of PFM 33, 34, &amp; 35</td>
</tr>
<tr>
<td>7</td>
<td>Debris blockage reduces spillway capacity and causes overtopping</td>
<td>Insignificant</td>
<td>Not carried forward</td>
<td>Not carried forward</td>
</tr>
<tr>
<td>8</td>
<td>Failure to open a penstock gate reduces capacity through the penstocks and causes overtopping</td>
<td>Potentially Significant</td>
<td>Part of PFM 33, 34, &amp; 35</td>
<td>Part of PFM 33, 34, &amp; 35</td>
</tr>
<tr>
<td>9</td>
<td>Penstock overflow leads to scour in the penstock area to breach due to sliding</td>
<td>Insignificant</td>
<td>Insignificant</td>
<td>Not carried forward</td>
</tr>
<tr>
<td>10</td>
<td>Monolith sliding</td>
<td>Potentially Significant</td>
<td>Part of PFM 33, 34, &amp; 35</td>
<td>Part of PFM 33, 34, &amp; 35</td>
</tr>
<tr>
<td>11</td>
<td>Apron displacement leads to increase scour and sliding potential</td>
<td>Potentially Significant</td>
<td>Part of PFM 33</td>
<td>Part of PFM 33</td>
</tr>
<tr>
<td>12</td>
<td>Sluice gate failure</td>
<td>Insignificant</td>
<td>Not carried forward</td>
<td>Not carried forward</td>
</tr>
<tr>
<td>13</td>
<td>Overtopping of the stilling basin training walls lead to scour and breach due to sliding</td>
<td>Potentially Significant</td>
<td>Not carried forward</td>
<td>Not carried forward</td>
</tr>
<tr>
<td>14</td>
<td>Failure of the Drift and debris gates</td>
<td>Insignificant</td>
<td>Not carried forward</td>
<td>Not carried forward</td>
</tr>
<tr>
<td>15</td>
<td>Monolith Overturning</td>
<td>Potentially Significant</td>
<td>Part of PFM 33, 34, &amp; 35</td>
<td>Part of PFM 33, 34, &amp; 35</td>
</tr>
<tr>
<td>16</td>
<td>Cavitation of the apron leads to PFM #11</td>
<td>Potentially Significant</td>
<td>Part of PFM 33</td>
<td>Part of PFM 33</td>
</tr>
<tr>
<td>17</td>
<td>Hydraulic Jacking of the stilling basin apron leads to PFM #11</td>
<td>Potentially Significant</td>
<td>Part of PFM 33</td>
<td>Part of PFM 33</td>
</tr>
<tr>
<td>18</td>
<td>Failure of the stilling weir due to sliding leads to scour causing breach of monolith due to sliding</td>
<td>Potentially Significant</td>
<td>Part of PFM 33</td>
<td>Part of PFM 33</td>
</tr>
<tr>
<td>19</td>
<td>Loss of load in anchor causes breach due to sliding</td>
<td>Insignificant</td>
<td>Not carried forward</td>
<td>Not carried forward</td>
</tr>
<tr>
<td>20</td>
<td>Sliding of training walls leads to scour and breach</td>
<td>Potentially Significant</td>
<td>Part of PFM 33</td>
<td>Part of PFM 33</td>
</tr>
<tr>
<td>21</td>
<td>Ogee cavitation</td>
<td>Insignificant</td>
<td>Not carried forward</td>
<td>Not carried forward</td>
</tr>
<tr>
<td>22</td>
<td>Internal sliding along internal plane in a pier</td>
<td>Insignificant</td>
<td>Not carried forward</td>
<td>Not carried forward</td>
</tr>
<tr>
<td>23</td>
<td>Sliding or overturning along an internal plane in the Assembly Bay or M-45</td>
<td>Potentially Significant</td>
<td>Not carried forward</td>
<td>Not carried forward</td>
</tr>
<tr>
<td>24</td>
<td>Internal instability in monoliths leading to partial displacement of monoliths</td>
<td>Potentially Significant</td>
<td>Not carried forward</td>
<td>Not carried forward</td>
</tr>
<tr>
<td>25</td>
<td>Motor Control Center fails causing all gates and gallery sump pumps to be inoperable leading to breach due to sliding</td>
<td>Potentially Significant</td>
<td>Part of PFM 33, 34, &amp; 35</td>
<td>Part of PFM 33, 34, &amp; 35</td>
</tr>
<tr>
<td>26</td>
<td>Bridge on the crest fails and causes blockage of the spillway that causes overtopping leading to breach due to sliding</td>
<td>Insignificant</td>
<td>Not carried forward</td>
<td>Not carried forward</td>
</tr>
<tr>
<td>27</td>
<td>Cavitation in the penstock leads to undermining and sliding</td>
<td>Insignificant</td>
<td>Not carried forward</td>
<td>Not carried forward</td>
</tr>
<tr>
<td>28</td>
<td>Debris or Ice Loading causes a structural failure of the crest gates</td>
<td>Insignificant</td>
<td>Not carried forward</td>
<td>Not carried forward</td>
</tr>
<tr>
<td>29</td>
<td>Sliding failure due to earthquake loading</td>
<td>Insignificant</td>
<td>Not carried forward</td>
<td>Not carried forward</td>
</tr>
<tr>
<td>30</td>
<td>Seismic failure of penstock gates</td>
<td>Insignificant</td>
<td>Not carried forward</td>
<td>Not carried forward</td>
</tr>
<tr>
<td>31</td>
<td>Structural failure leads to inability to open 1 or more crest gates.</td>
<td>Potentially Significant</td>
<td>Part of PFM 33, 34, &amp; 35</td>
<td>Part of PFM 33, 34, &amp; 35</td>
</tr>
<tr>
<td>32</td>
<td>Overtopping nappe creates a harmonic resonance which leads to internal instability.</td>
<td>Potentially Significant</td>
<td>Not carried forward</td>
<td>Not carried forward</td>
</tr>
<tr>
<td>33</td>
<td>Spillway Monolith Instability</td>
<td>Further Evaluation</td>
<td>Significant</td>
<td>Significant, Not Actionable</td>
</tr>
<tr>
<td>34</td>
<td>Non-overflow Monolith Instability</td>
<td>Further Evaluation</td>
<td>Significant</td>
<td>Significant, Not Actionable</td>
</tr>
<tr>
<td>35</td>
<td>Abutment Monolith Instability</td>
<td>Further Evaluation</td>
<td>Significant</td>
<td>Significant, Not Actionable</td>
</tr>
</tbody>
</table>
Significant failure modes were foundation stability of the dam and abutments, and structural stability of the spillway stilling basin. DSOG classified Bluestone Dam as a DSAC 2 in December 2008.

Developed Initial List of Potential Failure Modes (PFM) 
Completed a Report
Multiple comments received on the report triggered a reevaluation.

Facilitated PFMA
Developed the BCRA Report
35 Failure Modes Evaluated

Finalized BCRA Report
Presented to DSOG in July 2013
Directed to initiate a Dam Safety Modification Study
Explore the needs to update H&H
Team began revising the H&H (IDF, PMP, & PMF)

Due to H&H findings (increased PMF elevation, decreased PMP, and decreased PMF frequency) it triggered a reevaluation of the 2012 PFMA.

No new failure modes determined except PFM 9 has significant uncertainty. Still determined it is not a risk driver.
Presented to the vertical team in March 2016 and gained concurrence from RMC and HQ. PFM 33 is the only actionable failure mode.
### 4.3.1. PFM 33 - Spillway Monolith Instability

This PFM is the primary risk driver and consist of a combination of multiple PFMs that were identified in the PFMA. This PFM is described as follows:

> Hydrologic event generates significant inflows causing pool to rise to a level which is influenced by gate operability (PFM 6, 8, 25, and 31). Uplift acting on a failure plane at or below the structure base increases which could be influenced by flooding of the gallery (PFM 3). Discharges through the primary outlet works reduce passive resistance by flushing out tailwater and scour of the passive rock wedge (PFM 4) which could be influenced by the failure of one or more structures (PFM 11, 16, 17, 18, and 20) within the stilling basin. Two-dimensional driving forces acting on the spillway exceed resisting stabilizing forces and movement of one or more monoliths initiates. Intervention is unsuccessful. This initial movement begins to transfer load to adjacent monoliths and cause deformation of the prestressed anchors. Increased stresses in the anchors induced by this movement cause anchors to begin failing which results in additional excess driving forces which continue to be transferred to adjacent monoliths through additional downstream movement and engaging more monoliths. This process continues until a shallow arch forms across a portion of the dam. Deformation and anchor failure continues until crushing of the concrete or failure in the abutment occurs causing a failure of the arch and multiple monoliths abruptly displace a significant distance downstream (PFM 10 and 15) leading to breach and loss of pool.

(US Army Corps of Engineers, Bluestone Dam Baseline Condition Risk Assessment, 2013)

### 4.3.2. PFM 34 - Non-overflow Monolith Instability

This PFM is a risk driver; however, the risk has been determined to be below tolerable risk guidelines. Taking into account uncertainty, the team decided to retain it for additional risk reduction opportunities considering ALARP (as low as reasonably practicable) principles. As part of the formulation process, the PDT considered completing the partially built parapet wall as measure for an ALARP consideration. Prior to the quantitative with-project risk assessment, the team thought it may reduce risk further below tolerable risk guidelines; however, it was determined it did little to reduce the overall risk (Additional details can be found in Section 5.2.6.4.1). The description of this PFM is as follows:

> Hydrologic event generates significant inflows causing pool to rise to a level which is influenced by gate operability (PFM 6, 8, 25, and 31). Uplift acting on a failure plane at or below the structure base increases which could be influenced by flooding of the gallery (PFM 3). Flows over the crest of the dam scour materials at the downstream toe, thereby reducing passive resistance to monolith stability (PFM 2b). Two-dimensional driving forces acting on the non-overflow section of the dam exceeds resisting stabilizing forces and movement of one or more monoliths initiates. Intervention is unsuccessful. This initial movement begins to transfer load to adjacent monoliths and cause deformation of the prestressed anchors. Increased stresses in the anchors induced by this movement cause anchors to begin failing which results in additional excess driving forces which continue to be transferred to adjacent monoliths through additional downstream movement and engaging more monoliths. This process continues until a shallow arch forms across a large portion of the dam. Deformation and anchor failure continues until crushing of the concrete or failure in the abutment occurs causing a failure of the arch and multiple monoliths abruptly displace significant distance downstream (PFM 10 and 15) leading to breach and loss of pool.

(US Army Corps of Engineers, Bluestone Dam Baseline Condition Risk Assessment, 2013)
4.3.3. PFM 35 - Abutment Monolith Instability

This PFM is a risk driver, but similar to PFM 34, the evaluation proved it is below tolerable risk guidelines. The description of this PFM is as follows:

*Hydrologic event generates significant inflows causing pool to rise to a level which is influenced by gate operability (PFM 6, 8, 25, and 31). Uplift acting on a failure plane at or below the structure base increases which could be influenced by flooding of the gallery (PFM 3). Discharge that flows over the crest of the dam scours materials at the downstream toe, thereby reducing passive resistance to monolith stability (PFM 2a). Two-dimensional driving forces acting on the abutment monoliths exceed resisting stabilizing forces and movement of one or more monoliths initiates. Intervention is unsuccessful. This initial movement begins to transfer load to adjacent monoliths and cause deformation of the prestressed anchors. Increased stresses in the anchors induced by this movement cause anchors to begin failing which results in additional excess driving forces which continue to be transferred to adjacent monoliths through additional downstream movement and engaging more monoliths. This process continues until a shallow arch forms across a portion of the dam. Deformation and anchor failure continues until crushing of the concrete or failure in the abutment occurs causing a failure of the arch and multiple monoliths abruptly displace significant distance downstream (PFM 10 and 15) leading to breach and loss of pool.* (US Army Corps of Engineers, Bluestone Dam Baseline Condition Risk Assessment, 2013)

Similar to PFM 34, an option was considered to complete the partially built parapet wall as an ALARP (as low as reasonably practicable) consideration; however, it was determined it did not reduce the risk of this failure mode further below tolerable risk guidelines.

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**Figure 4-3: Significant Failure Modes at Bluestone Dam**

4.4. System Response

Considering that the only actionable PFM is PFM 33, the details associated with the system response curves for PFM 34 and PFM 35 are not discussed. For additional information see the Risk Assessment Technical Summary Report (US Army Corps of Engineers, Bluestone Dam Risk Assessment Technical Summary Report, 2016). This was confirmed at a vertical team meeting held in Pittsburgh, PA in March 2016. A memorandum was created summarizing the conclusions and outcomes of this meeting from the LRH DSO to the Director of the Risk Management Center (Martin, PE, PMP, 2017).

4.4.1. PFM 33 – Spillway Monolith Instability

The response for this PFM is driven by the total discharge through the primary stilling basin from both the crest gates and the sluices. The key initiating mechanism for this PFM is scour and loss of passive resistance.
to sliding stability for the dam monoliths. One of the critical components of this mechanism was the potential loss of the upstream stilling basin apron since this would expose erodible foundation rock to the plunging jet of flows coming down the face of the ogee spillway crest. This was due to the potential of the joint between the apron and the toe of the dam to open up creating stagnation pressures under the slab jacking and displacing it with the high velocity flows. Figure 4-4 shows scour and failure given displacement of the upstream apron. Breach of the stilling weir was not assessed as part of this PFM for the 2013 BCRA but recent analysis shows its stability would be compromised at discharges less than that which threaten the apron. Based on previous physical modeling, loss of the weir would lead to similar conditions that were expected to cause displacement of the apron. Additional analysis also indicated the apron could displace at lower discharges than those assumed for the 2013 BCRA. Therefore, the lower range of probabilities for apron displacement were adjusted to account for potential breach of the weir and updated apron assessment.

Figure 4-5 is the event tree used to determine the probability of breach for a given loading condition. The first level is the elicited probability of scour downstream of the apron for a given discharge and the second node is the elicited probability of displacement of the apron. It was determined the loss of the apron would lead to excessive scour at the toe due to the high velocity jet discharging through the crest gates. The key evidence for the scour probabilities was scour studies based on interpretations of movable bed geometry observed in a physical model by scour experts. The third level was the conditional probability of two-dimensional instability given the load and scour conditions. This was calculated as the probability of having a sliding factor of safety below 1.0 using a Monte Carlo simulation; which consisted of a probabilistic two-dimensional limit equilibrium analysis that considered uncertainty in various key parameters. The final level was the elicited probability of breach given two-dimensional instability considering three-dimensional effects that would allow for additional load transfer into adjacent structures and foundation. Details of these assessments can be found in the 2013 BCRA Report.
The BCRA assessed this failure mode for a range of discharges up through a total spillway and sluice discharge of approximately 860,000 cfs but with the new hydrologic hazard assessment the maximum spillway and sluice discharge is now approximately 980,000 cfs. Rather than trying to reconvene the original panel of experts to elicit the response at this higher discharge, the District determined the most efficient path forward was to extrapolate the response out to this new, higher discharge value. Given the adjusted conditional probability of breach at the 860,000 cfs was already high (9.5E-01) and additional discharge would only increase this risk, how this extrapolation is completed has very little effect on the overall risk for this failure mode. Conversely, the very remote probability of seeing these loads based on the hydrologic hazard curve would result in negligible change to the overall risk estimate.

The final system response curves and how they compare to the BCRA are shown in Table 4-3 and Figure 4-6. The combination of the adjusted, which takes into account weir instability and updated apron assessment, and extrapolated curves were used for the ECRA and FWAC assessments.
Figure 4-5: Event Tree used to determine the Probability of Breach for a Given Loading Condition
Table 4-3: PFM 33 – Spillway Monolith Instability System Response Table

<table>
<thead>
<tr>
<th>Spillway (M42)</th>
<th>Discharge</th>
<th>SRP</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>967721</td>
<td>9.990E-01</td>
</tr>
<tr>
<td></td>
<td>867700</td>
<td>9.496E-01</td>
</tr>
<tr>
<td></td>
<td>699800</td>
<td>4.358E-01</td>
</tr>
<tr>
<td></td>
<td>560100</td>
<td>2.955E-02</td>
</tr>
<tr>
<td></td>
<td>430000</td>
<td>6.444E-04</td>
</tr>
<tr>
<td></td>
<td>380000</td>
<td>1.515E-04</td>
</tr>
<tr>
<td></td>
<td>350000</td>
<td>1.262E-04</td>
</tr>
<tr>
<td></td>
<td>140000</td>
<td>1.530E-07</td>
</tr>
<tr>
<td></td>
<td>72000</td>
<td>1.000E-12</td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>1.000E-12</td>
</tr>
</tbody>
</table>

Figure 4-6: PFM 33 – Spillway Monolith Instability System Response Curve
4.5. Consequence Analysis Overview

Consequences were calculated based on incremental risk between the breach and non-breach scenarios. In other words, incremental consequences are defined as the difference between consequences of inundation in a non-failure event and consequences in the failure event. An overview of the consequence analysis is discussed in Section 4.5; however, additional details can be found in Appendix J.

4.5.1. Dam Breach Analysis and Inundation Mapping

Dam failure modeling developed for the BCRA was updated to include additional understanding of breach parameters, updated hydrology, and key DSMS elevations, and a detailed hydraulic model geometry for the pool and inline structure (i.e. Bluestone Dam). The dam failure model extents are from Bluestone Dam to Greenup Locks and Dam on the Ohio River (237 river miles downstream). The area upstream of Bluestone Dam was modeled as a storage area and a headwater model was included which dynamically routed the revised IDF through reaches upstream of the dam (see Figure 4-7 for a map). A set of 11 pool elevations were modeled ranging from the summer (normal) low pool (EL 1,409-ft) to the maximum high pool (EL 1,553-ft) representing the minimum and maximum operating conditions. Some of the pools included multiple operating scenarios, including those for the existing and future without action conditions, leading to a total of 16 non-failure scenarios. Three pools (EL 1,530-ft, EL 1,525-ft, and EL 1,520-ft) were evaluated for future without action condition, which limits outflows up to a pool of EL 1,535-ft, above which gates are fully opened. Table 4-4 lists a summary of all non-failure model scenarios. For each of these scenarios, a non-failure and failure condition were assumed and modeled. Breach initiation was assumed to occur at the modeled peak pool elevation reached at Bluestone Dam. The best estimate breach width of 11 spillway monoliths (from the 2013 BCRA elicitations) was assumed for all failure models. A reasonable low case was not specifically determined but was not expected to be significantly lower than the best estimate. In addition, to determine sensitivity to breach width, a reasonable high case of 22 monoliths (the full width of the spillway) was also modeled for some scenarios. This larger breach width of double the width resulted in about a 50% increase in peak discharge at the dam but only a 20% increase in Charleston, the primary consequence center. Although not modeled, a smaller breach width is expected to not have an appreciable impact on the consequences.

Table 4-4: Hydrologic Loading Condition & Target Pool Elevation

<table>
<thead>
<tr>
<th>Hydrologic Loading Condition &amp; Target Pool Elevation (feet NGVD29)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max High Pool (PMF)</td>
</tr>
<tr>
<td>Intermediate Pool for Consequences with 300k cfs limitation (1525)</td>
</tr>
<tr>
<td>Intermediate Pool for Consequences (1548)</td>
</tr>
<tr>
<td>Top of Active Storage, Gates Open (1520)</td>
</tr>
<tr>
<td>Top of Proposed Parapet Wall (1543)</td>
</tr>
<tr>
<td>Top of Active Storage, Gates Half Open (1520)</td>
</tr>
<tr>
<td>Top of Existing Rails (1539)</td>
</tr>
<tr>
<td>Top of Active Storage, Gates Closed (1520)</td>
</tr>
<tr>
<td>Existing Top of Dam (1536)</td>
</tr>
<tr>
<td>Top of Active Storage, Gates Open with 300k cfs limitation (1520)</td>
</tr>
<tr>
<td>Intermediate Pool for Consequences (1530)</td>
</tr>
<tr>
<td>Security Scenario (1434)</td>
</tr>
<tr>
<td>Intermediate Pool for Consequences with 300k cfs limitation (1530)</td>
</tr>
<tr>
<td>Normal High Pool (1411)</td>
</tr>
<tr>
<td>Intermediate Pool for Consequences (1525)</td>
</tr>
<tr>
<td>Normal Low Pool (1409)</td>
</tr>
</tbody>
</table>
4.5.2. Life Loss Consequences
The HEC-FIA software was used to estimate consequences associated with potential breach and non-breach scenarios of Bluestone Dam. HEC-FIA estimates flooding damage similar to the methods used by the older HEC-FDA software, but has the additional capability to estimate potential life loss from flood scenarios. The life loss methodology in HEC-FIA is based on the methodology developed by Utah State University’s Institute for Dam Safety Risk Management. The process of computing loss of life within FIA is to identify the population at risk (PAR) from a given event and then divide this PAR into those cleared from the danger area, those caught evacuating, and those not mobilized. This division is based on a host of factors, including warning time relative to the flood wave arrival time, mobilization, and distance to a safe zone (Figure 4-7). Those who do not escape the hazard area are subjected to fatality rates that are a function of evacuation status, depth, foundation height, and structure height. The consequence data presented in this report were generated using HEC-FIA version 3.0 Beta (from November 2015).

The parameters used within HEC-FIA to evaluate life loss are generally different between the existing condition and FWAC. The parameters that changed between those scenarios include the population per structure, the warning diffusion curve, and the mobilization curves. Of these parameters, only changes in the mobilization curves had a significant impact on the life loss estimates. Detailed discussion on how these parameters were developed can be found in section 3.2.3 of this report.
If Bluestone Dam Failed at Pool Elevation 1535 feet

- **Point Pleasant, WV / Gallipolis, OH**
  - NWS Flood Stage Elevation (feet): 553.0
  - Maximum Elevation (feet): 579.1
  - Arrival Time after Failure (hours): 22.0
  - Time to Peak Elevation (hours): 45.5
  - Recession Time (hours): 71.5

- **Charleston, WV**
  - NWS Flood Stage Elevation (feet): 584.3
  - Maximum Elevation (feet): 635.7
  - Arrival Time after Failure (hours): 6.9
  - Time to Peak Elevation (hours): 12.8
  - Recession Time (hours): 66.1

- **Kanawha Falls, WV**
  - NWS Flood Stage Elevation (feet): 648.2
  - Maximum Elevation (feet): 721.2
  - Arrival Time after Failure (hours): 3.2
  - Time to Peak Elevation (hours): 6.2
  - Recession Time (hours): 64.4

- **Parkersburg, WV**
  - NWS Flood Stage Elevation (feet): 596.8
  - Maximum Elevation (feet): 593.8
  - Arrival Time after Failure (hours): N/A
  - Time to Peak Elevation (hours): N/A
  - Recession Time (hours): N/A

- **Ashland, KY / Ironton, OH**
  - NWS Flood Stage Elevation (feet): 532.2
  - Maximum Elevation (feet): 568.2
  - Arrival Time after Failure (hours): 39.3
  - Time to Peak Elevation (hours): 67.7
  - Recession Time (hours): 51.4

- **Huntington, WV**
  - NWS Flood Stage Elevation (feet): 539.5
  - Maximum Elevation (feet): 555.5
  - Arrival Time after Failure (hours): 37.7
  - Time to Peak Elevation (hours): 63.0
  - Recession Time (hours): 53.9

- **Hinton, WV**
  - NWS Flood Stage Elevation (feet): 1369.8
  - Maximum Elevation (feet): 1426.4
  - Arrival Time after Failure (hours): 0.2
  - Time to Peak Elevation (hours): 1.2
  - Recession Time (hours): 43.1

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Figure 4-7: Stakeholder Communication Graphic Developed by Huntington District

June 2017
The average incremental life loss from a breach of Bluestone Dam ranges from around 600 to 6,300 in the existing condition and from 400 to 2,800 in the FWAC, depending on the scenario. The largest incremental life loss occurs from a breach with the reservoir at the top of active storage and the spillway crest gates closed because there is less warning opportunity (no warnings based on spillway flow prior to breach). Life loss is typically higher during the day because commercial and industrial buildings, of which there are a significant amount of in the Charleston area, have many more people in them during the daytime than they do during the nighttime.

Over 80% of the life loss at the PMF breach event (EL 1,553-ft) is expected to occur in Kanawha County, which is about seven hours downstream from Bluestone Dam. At the PMF breach event (EL 1,552-ft), the average depth of flooding on 60,000 structures in Kanawha County is about 40 feet, compared to an average depth of about 32 feet in the PMF non-breach scenario. Mobilization of the 106,000 PAR in Kanawha County is expected to approach 95% in both FWAC breach and non-breach scenarios, which is the average maximum mobilization. Additional life loss consequence details can be found in Appendix J. Existing condition consequence results are also located in the Appendix J; they were not included in the main report as they are not used for plan comparison or risk analysis.

Table 4-5: FWAC Life Loss Averages and Ranges (Elevations are shown in NAVD 88)
The primary driver of life loss associated with Bluestone Dam is the high flood depths in Charleston associated with both breach and non-breach spillway discharges. Because the depths are so high, any population that does not mobilize would have a high fatality rate. Since the population at risk is on the order of magnitude of 100,000 people, even if 95% of the population can successfully mobilize and evacuate that still leaves the potential for several thousand people to remain and be subjected to extreme flood depths with low rates of survivability. The majority of the life loss is estimated to occur in and around Charleston due to the large amount of PAR in that area. The variation in life loss estimates depicted by the ranges primarily represents the uncertainty sampling around the estimated maximum mobilization rates used in the analysis. A higher maximum mobilization rate significantly reduces life loss while a lower maximum increases it.

Most of the scenarios involve spillway flow prior to the breach, which would trigger evacuation warnings well in advance of a breach. Because of this early warning and the distance between the dam and the high density population area around Charleston, variations in warning issuance time and warning diffusion have very little impact on the potential life loss. Basically, getting the warning out isn’t a problem; rather, life loss is dependent on how effective that warning is in convincing people to evacuate and the overall capability of emergency services to facilitate that evacuation. The range represents the uncertainty around those estimates, which are discussed in Section 3.2.3 of this report.

The life loss estimates in the table above are inputs for DAMRAE, which combines them with the system response probabilities to determine the annual life loss values used to assess risk.

4.5.3. Economic Consequences
In addition to loss of life, a major dam breach has the potential to inflict massive economic damage downstream of Bluestone Dam. The consequences analysis in the risk assessment quantified some of the potential negative economic impacts, including: urban damage, infrastructure damage, and lost economic benefits as described in the following sections. The economic damages and lost benefits used for this analysis were considered to remain the same for both existing and FWAC conditions because the amount of buildings downstream and the amount of benefits provided by the project are not expected to change in any significant manner under the FWAC condition. Additional details can be found in Appendix J.

4.5.3.1. Urban Damages (Also Known As Property Damage)
Economic damages are inclusive of estimated damages to structures, contents, and vehicles. These values were calculated using the HEC-FIA model and the associated structure inventory. These damages are likely lower than they would be in reality as many are based on tax assessed values which are typically lower than actual replacement values. The calculated range of incremental economic damages is $700M to $4.8B and a maximum total economic damage due to breach at the PMF of $9.5B.
### Table 4-6: Economic Consequences of a Breach at Bluestone Dam (Elevations are shown in NAVD 88)

<table>
<thead>
<tr>
<th>HEC-FIA Scenario</th>
<th>Breach # Structures</th>
<th>Breach Damages ($)</th>
<th>Non-Breach # Structures</th>
<th>Non-Breach Damages ($)</th>
<th>Incremental # Structures</th>
<th>Incremental Damages ($)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1552 Breach - 22 Monoliths</td>
<td>88,615</td>
<td>$9,639</td>
<td>83,070</td>
<td>$8,831</td>
<td>5,545</td>
<td>$808</td>
</tr>
<tr>
<td>1535 Breach - 22 Monoliths</td>
<td>82,973</td>
<td>$8,659</td>
<td>75,466</td>
<td>$7,665</td>
<td>7,507</td>
<td>$994</td>
</tr>
<tr>
<td>1552 Breach - Abutment</td>
<td>87,499</td>
<td>$9,532</td>
<td>83,070</td>
<td>$8,831</td>
<td>4,429</td>
<td>$701</td>
</tr>
<tr>
<td>1542 Breach - Abutment</td>
<td>84,309</td>
<td>$8,913</td>
<td>78,467</td>
<td>$8,064</td>
<td>5,842</td>
<td>$849</td>
</tr>
<tr>
<td>1538 Breach - Abutment</td>
<td>82,971</td>
<td>$8,691</td>
<td>76,831</td>
<td>$7,838</td>
<td>6,140</td>
<td>$853</td>
</tr>
<tr>
<td>1552 Breach - 11 Monoliths</td>
<td>87,303</td>
<td>$9,541</td>
<td>83,070</td>
<td>$8,831</td>
<td>4,233</td>
<td>$710</td>
</tr>
<tr>
<td>1542 Breach - 11 Monoliths</td>
<td>84,141</td>
<td>$8,890</td>
<td>78,467</td>
<td>$8,064</td>
<td>5,674</td>
<td>$826</td>
</tr>
<tr>
<td>1538 Breach - 11 Monoliths</td>
<td>82,794</td>
<td>$8,668</td>
<td>76,831</td>
<td>$7,838</td>
<td>5,963</td>
<td>$830</td>
</tr>
<tr>
<td>1535 Breach - 11 Monoliths</td>
<td>81,706</td>
<td>$8,493</td>
<td>75,466</td>
<td>$7,665</td>
<td>6,240</td>
<td>$828</td>
</tr>
<tr>
<td>1529 Breach - 11 Monoliths</td>
<td>74,255</td>
<td>$7,527</td>
<td>66,196</td>
<td>$6,352</td>
<td>8,059</td>
<td>$1,175</td>
</tr>
<tr>
<td>1524 Breach - 11 Monoliths</td>
<td>65,858</td>
<td>$6,321</td>
<td>54,417</td>
<td>$4,309</td>
<td>11,441</td>
<td>$2,012</td>
</tr>
<tr>
<td>1519 Gates Half Open Breach - 11 Monoliths</td>
<td>57,185</td>
<td>$4,988</td>
<td>18,815</td>
<td>$363</td>
<td>38,370</td>
<td>$4,625</td>
</tr>
<tr>
<td>1519 Gates Closed Breach - 11 Monoliths</td>
<td>56,122</td>
<td>$4,785</td>
<td>1,072</td>
<td>$23</td>
<td>55,050</td>
<td>$4,762</td>
</tr>
<tr>
<td>1433 Breach - 11 Monoliths</td>
<td>1,216</td>
<td>$1,525</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1410 Breach - 11 Monoliths</td>
<td>421</td>
<td>$514</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**4.5.3.2. Infrastructure Damage**

The state capital and state government building complex are located on the southeastern side of downtown next to the river. Most of the economic activity in the area is oriented toward chemical production and coal mining. The chemical industry supports a total of 22,500 jobs in Kanawha and Putnam Counties, which represents almost 60% of all manufacturing jobs in the greater Kanawha Valley.

A failure of Bluestone Dam could potentially result in significant impacts to a large number of critical infrastructure and key resource assets found in the Kanawha Valley. The greatest perceived threat stems from potential hazardous chemical spills near the City of Charleston that could put human health, property, and water supplies at risk for an unknown distance downstream and duration of time. Some chemicals could become airborne, posing risk from inhalation that would have effects outside of the inundation area. The chemicals could result in flooded structures requiring decontamination after flooding, and it could also put rescue workers at higher risk. There are three major sites that will be flooded in which chemical production is concentrated: the DuPont Belle plant, the Dow South Charleston site, and the Bayer Crop Science Institute site, all located in and around the City of Charleston. There are also a number of potable water and sewage treatment plants, which contain chlorine, along the entire New and Kanawha River basin. Flooding of these industrial areas could pose a special hazard. Both gaseous emissions and widespread water contamination could result from flooding due to dam failure. Many of the chemicals used in the valley are highly toxic in low concentrations including chlorine, organic chemicals, antifreeze, pesticides, herbicides, and the chemical precursors of the these compounds. It is
reasonable to assume that a catastrophic flood could cause significant chemical impacts to terrestrial habitat, aquatic habitat, and susceptible communities along the entire river valley.

There’s also the potential loss of John Amos Power Plant, one of the region’s largest power suppliers, and up to five large hospitals comprising the Charleston Area Medical Center that would likely be forced to evacuate. Damage to USACE navigation structures (London, Marmet, and Winfield lock and dams) could cause loss of navigation pools along the Kanawha River that would severely impact the ability to transport coal and other critical goods. Regional rail assets would probably be damaged as well. Most of the major industry, businesses, services, and government buildings in Charleston would also likely be significantly impaired.

4.5.3.3. Lost Benefits
Among the authorized project purposes of Bluestone Dam are flood risk management and recreation. Each of these project purposes would be impacted in the event of a dam failure, as described below. Benefits foregone in the event of a dam failure represent benefits provided by the dam as a result of its continued operation, and lost in the event of a dam failure. Foregone flood risk management benefits represent the aggregated average yearly reductions in flood damages provided by the dam for the anticipated repair period following a failure. Recreational benefits foregone represent the monetized value of recreational opportunities provided by the presence of the dam that would no longer be available or as worthwhile in the event of a dam failure. Bluestone Dam was originally authorized for hydropower and penstocks were incorporated into the design and construction but turbines were never added. Part of the consequences of a breach include the loss of the benefits associated with future operation of the dam, although they are relatively small in comparison with the flood damage caused by a breach. Additional details and values of the lost benefits associated with a dam breach can be found in Section 2.7.1.

4.5.4. Environmental Consequences
The environmental consequences from a dam breach for both the ECRA and FWAC would result in catastrophic and significant long term environmental consequences to the downstream aquatic, terrestrial, riparian, and cultural resources due to extreme scour and destruction. Additional details for each resource can be found in Chapter 5 of Appendix K.

4.6. Risk Assessment Results
The computer program Dam Safety Risk Analysis Engine (DAMRAE) Version 2.1.1.4 was used for performing event tree risk model computations for dam safety risk analysis. The program integrates input relationships for load, response, and consequence by method of finite difference to calculate risk. Calculations are based on event trees that are created by the user. For this project, a single event tree was created to calculate project risk associated with hydrologic loading.

To show a potential range of uncertainty associated with the loading and consequences, runs with reasonable highs and lows on both inputs were included along with the expected values. Output was then post-processed in a spreadsheet to generate plots used to compare the risks against USACE established tolerable limits.

4.6.1. Intervention
An estimation of intervention is not included in this assessment as it is expected to be virtually impossible. Failure is driven by scour in the primary stilling basin after gates have been opened to pass inflows. During this release it is possible there would be some visual indication that scour has initiated. In this case, closing the gates may not be a good option as this would result in overtopping and likely failure of the gates, resulting in the same or greater release, or leading to earlier overtopping of the non-overflow and
abutment sections of the dam. There is also no technically feasible means of arresting the scour under the high velocity and turbulent flow during an event.

If breach of the dam did not result from flows through the primary spillway and the reservoir was expected to rise to levels resulting in overtopping of the non-overflow and abutment portions of the dam, there would be no feasible means of raising the effective top of dam.

4.6.2. Event Tree
The event tree used for this update is identical to that used for the BCRA. The event tree begins with a continuous branch, but instead of relating pool to exceedance probability, this branch relates inflow to exceedance probability. The next three branches are state functions that relate inflow to pool (Level 2), spillway discharge (Level 3), and total discharge (Level 4). This approach more accurately calculates risk associated with stilling basin erosion because the gated spillway allows for a broad range of discharge for a given pool of approximately EL 1,520-ft. The event tree is shown in Figure 4-8. The inputs for DAMRAE are included in the following sections and in Appendix D for the hydrologic loading.

![Event Tree Diagram]

Figure 4-8: Event Tree

4.6.3. Existing Condition Risk Assessment
The event tree previously discussed and shown in Figure 4-8 was used to estimate the project risk for three different scenarios to approximate a best estimate, a reasonable low, and a reasonable high. The system response probabilities used for each scenario are the same as reasonable highs and lows were not estimated. However, three different estimates for hydrologic loading curves and three sets of consequence tables were used to portray sensitivity to these inputs. Table 4-7 through Table 4-9 present the resulting Annual Probability of Failure (APF), Average Annual Life Loss (AALL), economic damages, and non-breath Annual Life Loss values for each scenario. Cells highlighted in yellow indicate results that are within one order of magnitude of tolerable risk guidelines and cells highlighted in red indicate those that are above tolerable risk guidelines. Figure 4-9 shows the risk estimate results for each failure mode as well as total project risk for the existing condition. The points represent best estimate values and the boxes envelope the reasonable high and reasonable low of the estimate to show sensitivity to loading and
consequences. This chart includes the loading and response associated with all gates operating, as well as with 3 spillway crest gates failing to open to capture the risks associated with the current reliability of the aged gate operating equipment. The sensitivity to loading and consequence boxes are only associated with the all gates operate scenario. Although not included, the sensitivity boxes for the 3 gate scenario are expected to shift similarly.

These estimates show that the existing condition risk from PFM 33 and the total risk exceed the tolerable risk limit for life safety as described in ER 1110-2-1156 with the best estimate being more than an order of magnitude greater than the limit. Although PFM 33 contributes over 95% of the total risk, PFM’s 34 and 35 are estimated to be in the Low Probability – High Consequence zone for the existing condition. For comparison purposes, these estimates were calculated assuming all gates were operational. There is a high likelihood that multiple gates would fail to open (55% for 2 to 4 gates) and changing the loading and response of the structure. Therefore, the risks were calculated for a scenario of 3 gates failing to operate, similar to what was done for the 2013 BCRA, using the best estimate for loading frequency and consequences. These results, and how they compare to the gates operable scenario, for APF and AALL are shown in Table 4-10 and Table 4-11. These risks show a significant increase in risk for both the spillway and overtopping failure modes but this increase is shown to be less than an order of magnitude and the risks associated with the overtopping PFMs remain below tolerable levels. However, these risks are likely to increase over time without significant rehabilitation.

Probability distributions of potential life loss are plotted in an F-N chart presented in Figure 4-10. Again, this plot shows risks above the tolerable risk limit for life safety.

The distribution of life loss associated with proper function (non-breach) of Bluestone Dam was also calculated in DAMRAE and is shown in Figure 4-11. Under normal operation conditions of the dam, spillway flows begin to exceed the downstream river channel capacity for many miles downstream of the project during events with about $5 \times 10^{-3}$ annual chance exceedance. This non-breach flooding will inundate a very large number of homes, businesses, and public infrastructure including major chemical manufacturing facilities in the Kanawha Valley. Even though warnings will be issued, some inhabitants will refuse to leave. Inundation is of sufficient depth to drown a large percentage of those who do not or cannot evacuate. Flooding of industrial areas during non-breach scenarios could pose a special hazard due to both gaseous emissions and widespread water contamination. Assessments of inundation levels show that major chemical plant infrastructure is flooded by 6 to 13 feet of water during non-breach scenarios at pools above flood control pool. It is reasonable to assume that a catastrophic flood could cause significant chemical impacts to terrestrial habitat, aquatic habitat, and susceptible communities along the entire river valley.

### Table 4-7: APF for Each Scenario for Bluestone Dam

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Annual Probability of Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Abutment (PFM 35)</td>
</tr>
<tr>
<td>Reasonable Low</td>
<td>6.81E-10</td>
</tr>
<tr>
<td>Best Estimate</td>
<td>4.30E-07</td>
</tr>
<tr>
<td>Reasonable High</td>
<td>1.89E-06</td>
</tr>
</tbody>
</table>
## Table 4-8: AALL for Each Scenario for Bluestone Dam

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Abutment (PFM 35)</th>
<th>Non-Overflow (PFM 34)</th>
<th>Spillway (PFM 33)</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reasonable Low</td>
<td>3.67E-07</td>
<td>1.97E-08</td>
<td>5.92E-04</td>
<td>5.92E-04</td>
</tr>
<tr>
<td>Best Estimate</td>
<td>4.61E-04</td>
<td>3.99E-05</td>
<td>2.15E-02</td>
<td>2.20E-02</td>
</tr>
<tr>
<td>Reasonable High</td>
<td>2.77E-03</td>
<td>2.55E-04</td>
<td>1.28E-01</td>
<td>1.31E-01</td>
</tr>
</tbody>
</table>

## Table 4-9: Economic Damages and Non-Breach Risks for Bluestone Dam

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Annual Incremental Economic Consequences</th>
<th>Non-Breach Annual Life Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reasonable Low</td>
<td>$1,777</td>
<td>2.79E-02</td>
</tr>
<tr>
<td>Best Estimate</td>
<td>$33,061</td>
<td>2.93E-01</td>
</tr>
<tr>
<td>Reasonable High</td>
<td>$140,351</td>
<td>2.19E+00</td>
</tr>
</tbody>
</table>

## Table 4-10: Effect on Gate Reliability on APF

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Abutment (PFM 35)</th>
<th>Non-Overflow (PFM 34)</th>
<th>Spillway (PFM 33)</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gates Operable</td>
<td>4.30E-07</td>
<td>3.77E-08</td>
<td>1.03E-05</td>
<td>1.08E-05</td>
</tr>
<tr>
<td>3 Gates Fail to Open</td>
<td>6.45E-07</td>
<td>5.54E-08</td>
<td>2.69E-05</td>
<td>2.76E-05</td>
</tr>
</tbody>
</table>

## Table 4-11: Effect of Gate Reliability on AALL

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Abutment (PFM 35)</th>
<th>Non-Overflow (PFM 34)</th>
<th>Spillway (PFM 33)</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gates Operable</td>
<td>4.61E-04</td>
<td>3.99E-05</td>
<td>2.15E-02</td>
<td>2.20E-02</td>
</tr>
<tr>
<td>3 Gates Fail to Open</td>
<td>8.18E-04</td>
<td>6.32E-05</td>
<td>9.87E-02</td>
<td>9.96E-02</td>
</tr>
</tbody>
</table>
Figure 4-9: ECRA F-N Chart

Orange Triangles are best estimates given 3 gates fail to operate
Blue Circles are best estimates given all gates operate
Dashed boxes indicate sensitivity to reasonable high/low loading frequency and consequences (all gates operate)

Risks are unacceptable, except in extraordinary circumstances

Societal tolerable risk limit for average annual life loss

Lower risks to a tolerable level informed by the ALARP considerations

PFM 33 - Spillway Monolith Instability
PFM 34 - Non-Overflow Monolith Instability
PFM 35 - Abutment Monolith Instability

Total
PFM 33
PFM 34
PFM 35
Low Probability - High Consequence Events

Average Incremental Life Loss, $\bar{N}$
Annual Probability of Failure (APF), $f$

Bluestone Dam – Final DSMR

June 2017
Lower risks to a tolerable level informed by the ALARP considerations

Societal tolerable risk limit

Risks are unacceptable, except in extraordinary circumstances

Figure 4-10: ECRA F-N Chart
Figure 4-11: ECRA Non-Breach F-N Chart
4.6.4. **Future without Federal Action Condition Risk Estimate**

As described in previous sections and Table 3-1, the future without Federal action condition (FWAC) will differ slightly from the existing condition for the project. The primary differences are the assumption of the operation of the dam with respect to the threshold discharge and the mobilization rates for the downstream public in the event of a significant flood. The same methodology and event tree were used in DAMRAE to calculate a risk estimate for the FWAC. The following tables and figures are the same as were presented above with the exception of not including tables that show the effects of spillway crest gate reliability. This is due to the assumption of spillway crest gate machinery being rehabilitated for the FWAC, resulting in much lower probability of multiple gates failing to open. Based on the results discussed in Section 3.2.3.2.2, without a rehabilitation of the gate operating equipment the risks associated with the overtopping PFMs are likely to exceed tolerable levels in the future. Table 4-15 is included to show a direct comparison of the existing and future best estimate results.

**Table 4-12: APF for Each Scenario for Bluestone Dam - FWAC**

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Annual Probability of Failure</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Abutment (PFM 35)</td>
<td>Non-Overflow (PFM 34)</td>
<td>Spillway (PFM 33)</td>
<td>Total</td>
</tr>
<tr>
<td>Reasonable Low</td>
<td>3.07E-10</td>
<td>7.60E-13</td>
<td>1.63E-07</td>
<td>1.63E-07</td>
</tr>
<tr>
<td>Best Estimate</td>
<td>4.53E-07</td>
<td>3.94E-08</td>
<td>6.91E-06</td>
<td>7.40E-06</td>
</tr>
<tr>
<td>Reasonable High</td>
<td>1.97E-06</td>
<td>1.82E-07</td>
<td>2.74E-05</td>
<td>2.95E-05</td>
</tr>
</tbody>
</table>

**Table 4-13: AALL for Each Scenario for Bluestone Dam - FWAC**

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Annual Incremental Life Loss</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Abutment (PFM 35)</td>
<td>Non-Overflow (PFM 34)</td>
<td>Spillway (PFM 33)</td>
<td>Total</td>
</tr>
<tr>
<td>Reasonable Low</td>
<td>8.46E-08</td>
<td>2.16E-10</td>
<td>6.73E-05</td>
<td>6.74E-05</td>
</tr>
<tr>
<td>Best Estimate</td>
<td>2.70E-04</td>
<td>2.32E-05</td>
<td>5.85E-03</td>
<td>6.14E-03</td>
</tr>
<tr>
<td>Reasonable High</td>
<td>1.90E-03</td>
<td>1.73E-04</td>
<td>3.71E-02</td>
<td>3.92E-02</td>
</tr>
</tbody>
</table>

**Table 4-14: Economic Damages and Non-Breach Risks for Bluestone Dam - FWAC**

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Annual Incremental Economic Consequences</th>
<th>Non-Breach Annual Life Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reasonable Low</td>
<td>$507</td>
<td>5.72E-03</td>
</tr>
<tr>
<td>Best Estimate</td>
<td>$15,038</td>
<td>9.76E-02</td>
</tr>
<tr>
<td>Reasonable High</td>
<td>$39,965</td>
<td>8.20E-01</td>
</tr>
</tbody>
</table>

**Table 4-15: Comparison of Existing and Future Total Risk Estimates**

<table>
<thead>
<tr>
<th>Scenario</th>
<th>APF</th>
<th>AALL</th>
<th>Non-Breach AALL</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>EC</td>
<td>FWAC</td>
<td>EC</td>
</tr>
<tr>
<td>Reasonable Low</td>
<td>5.04E-07</td>
<td>1.63E-07</td>
<td>5.92E-04</td>
</tr>
<tr>
<td>Best Estimate</td>
<td>1.08E-05</td>
<td>7.40E-06</td>
<td>2.20E-02</td>
</tr>
<tr>
<td>Reasonable High</td>
<td>4.54E-05</td>
<td>2.95E-05</td>
<td>1.31E-01</td>
</tr>
</tbody>
</table>
Figure 4-12: FWAC f-N Chart
Figure 4-13: FWAC F-N Chart
Figure 4-14: FWAC Non-Breach F-N Chart
4.7. Risk Assessment Conclusions

Figure 4-15 shows the relationship between breach, non-breach, and residual risk for both the ECRA and FWAC. The existing condition estimates are for those that include 3 gates failing to operate. The non-breach risks are characterized by the annual probability of having discharges from the dam that result in potential life loss. The residual risk is presented as the sum of the breach and non-breach risks as calculated by DAMRAE.

Based on the ECRA and FWAC risk assessment results, the only failure mode that is deemed actionable is the PFM 33. This conclusion is based on two assumptions:

- Any fix for the spillway would not transfer any additional risk to the non-overflow or abutment monoliths as it would not reduce the capacity of the spillway resulting in additional overtopping flows; and
- If the spillway fix is only designed to the top of the dam, the nature of this design would result in lower probability of failure for PFM 33 than that of the PFM 34 and PFM 35.

While PFM 34 and 35 are below tolerable risk guidelines, measures (parapet wall) were developed to address these PFMs for ALARP considerations.
Figure 4-15: Risk plot for breach, non-breach, and residual risk for both the existing and future conditions
4.8. Uncertainty
The three main components of the risk calculations, hydrologic loading, system response, and consequences, can have significant uncertainties. A brief discussion of these uncertainties and how they are portrayed is presented below.

4.8.1. System Response Probability (SRP) Uncertainty
Uncertainty with the system response primarily revolves around the fact that much of the assessment of hydraulic performance and scour are based on scaled physical modeling. Simplifications were also made to the scour geometry since the actual geometry could vary widely. There is also significant uncertainty with the response of the structure given the reservoir has remained virtually empty with only sporadic flood loading and a pool of record well below levels that are expected to cause instability. This required extensive extrapolation of potential uplift pressures and drain efficiency. The effects on uplift due to the installation and grouting of the anchors also had to be estimated given this construction is still on-going in addition to even more limited high pool events available to analyze instrumentation data with respect to this effect. However, some of this uncertainty is captured in the Monte Carlo analysis and the overall results outside of this should not result in a total risk variation of more than an order of magnitude. A more detailed discussion of these uncertainties is included in Chapter 9 of the 2013 BCRA Report. (US Army Corps of Engineers, Bluestone Dam Baseline Condition Risk Assessment, 2013)

Further physical modeling or analysis is not likely to reduce much, if any, of this uncertainty. The best way to reduce uncertainty in the system response is for the dam to experience loading events that exceed historical highs for reservoir elevation and discharge. In the absence of performance observations under record events, there is no reasonable means of reducing uncertainty that would influence the dam safety decision for Bluestone Dam.

4.8.2. Hydrology
There is much uncertainty in extreme flood hydrologic estimates; both in developing estimates of the PMP/PMF and establishing the extreme range of the hydrologic loading curve. A portion of this uncertainty is reducible through further study; but much of the uncertainty is due to natural variability and small sample size of observed extreme events. The uncertainty associated with the DSMS hydrology was evaluated either qualitatively through sensitivity analysis or quantified numerically from the assumed probability distribution.

4.8.2.1. PMP/PMF
The PMP values used in the current PMF estimate, evaluated by the USACE Extreme Storm Team, is a best estimate in accordance with USACE practice. The major uncertainties associated with PMP is the variation in the storm centering, spatial distribution of rainfall, temporal distribution of rainfall, and total rainfall depth. Sensitivity to the storm centering was evaluated and the location that results in the maximum inflow into the project was selected. The uncertainties associated with the spatial distribution, temporal distribution, and total depth were not assessed. The major uncertainties with the resulting PMF were evaluated through performing sensitivity analysis. Sensitivities evaluated include: loss rates, reach routing parameters, sub-basin unit hydrograph lag, antecedent pool, and gate operations. Of these, the sub-basin unit hydrograph lag and the loss rates had the largest contribution to uncertainty. It is likely that the largest contributor to uncertainty associated with the estimate of the PMF peak pool is the PMP spatial and temporal patterns.

Considering that the dam safety decision related to the Tentatively Selected Plan (TSP) is relatively insensitive to increases in IDF and a decrease of approximately 30% would be required for modification to the TSP, additional analysis of uncertainty does not appear supported at this time.
4.8.2.2. Hydrologic Hazard Curve
The DSMS decisions are mostly affected by the extreme end of the loading curve, at inflow events much greater than what has been recorded at the site. Therefore, the discussion on uncertainty of the hydrologic hazard curve is focused on this range of loading. The primary driver of the uncertainty in the estimate of probability of extreme events is the flow frequency curve. The computed uncertainty in the distribution associated with the sample size spans 3 orders of magnitude between the 95th percentile and the median curve near PMF levels. This analysis does not take into account the uncertainty associated with the selection of a parametric distribution for flow frequency. Secondary uncertainty factors in the hydrologic hazard curve include hydrograph shape, gate reliability, penstock operations, and critical flood duration. Tertiary uncertainty factors assessed were starting reservoir storage and downstream flood impacts on operation, as detailed in Appendix D.

Additional actions to reduce key uncertainties is not recommended for this decision document based on the already large uncertainties included in the hydrologic hazard characterization and the limited available analysis methods to reduce uncertainty and improve estimates. If additional federal investment is available in the future, consideration should be given to evaluating additional sources of uncertainty and an attempt to reduce those uncertainties should be made.

4.8.3. Consequence Uncertainty
The HEC-FIA model was run with uncertainty on several parameters. On all previous modeling on this dam, the most sensitive parameter was the mobilization curve, particularly the maximum rates. To capture the range of uncertainty, distributions were used for warning times, mobilization curves, and individual fatality rates. Initial tests showed that the model results were not overly sensitive to uncertainty on structure foundation heights, so no distribution was used around estimated foundation heights. Initial model runs also showed that average life loss estimates would generally converge within a tolerance of 5% of the mean within 40-50 simulations. Since each simulation took approximately 15 minutes and there were over 45 breach and non-breach scenarios to run (over 20 days of computer run time), between 40 and 70 simulations were run for each event scenario depending on how close the convergence was after the initial 40 simulations.

The most sensitive parameter, mobilization, is also likely the most uncertain as it tries to predict human behavior. Reduction of this uncertainty would be difficult. The sensitivity analysis showed that reduction of this uncertainty is not likely to influence the decision with the range of potential loss of life roughly within the same order of magnitude.

4.8.4. Uncertainty Analysis Results
The risk assessment previously discussed includes the evaluation of three different scenarios to approximate a best estimate, a reasonable low, and a reasonable high. Three different estimates for hydrologic loading curves and three sets of consequence tables were used to portray sensitivity to these inputs. Table 4-12 through Table 4-14 present the resulting Annual Probability of Failure (APF), Average Annual Life Loss (AALL), economic damages, and non-breach Annual Life Loss values for each scenario. Cells highlighted in yellow indicate results that are within one order of magnitude of tolerable risk guidelines and cells highlighted in red indicate those that are above tolerable risk guidelines. Figure 4-12 shows the risk estimate results for each failure mode as well as total project risk for the FWAC condition. The points represent the best estimate values and the boxes envelope the reasonable high and reasonable low of the estimate to show sensitivity to loading and consequences.
5. PLAN FORMULATION OVERVIEW

Chapter 5 summarizes the plan formulation process for the Bluestone DSMS. This chapter describes how risk management measures and plans were developed and formulated to meet the study objectives and the criteria and process used to recommend a TSP to address the identified dam safety issues. As discussed in Chapter 4, only one actionable PFM was identified for Bluestone Dam, PFM 33 – Spillway Monolith Instability. Therefore, formulation of RMPs focused on this PFM which drove incremental risk and warranted Federal action. The measures and plans were formulated to address the three vulnerabilities of PFM 33. Therefore the measures and plans developed address instability of the weir, displacement of the existing apron, and scour of the unlined stilling basin.

Risk management plans were formulated to meet the study objective of identifying and recommending a cost effective RMP to expeditiously reduce incremental risk to achieve USACE TRG as defined in ER 1110-2-1156, including ALARP considerations. In summary, ALARP considerations is defined as actions that should be taken to reduce risk below the tolerable risk limit until such actions are impracticable or not cost effective.

5.1. Previous Formulation Efforts based on 2013 BCRA

Plan formulation began with the brainstorming of risk management measures to address the dam safety issues (risks) based on the results of the 2013 BCRA. These measures can be standalone solutions (plans) or used as building blocks to combine multiple measures into a plan. Generally, measures were broken into two categories, structural and non-structural measures.

- Structural measures are ways to address risks by structurally modifying the dam (focuses on ways to address the annual probability of failure (APF) component of the risk equation);
- Non-structural measures are ways to address risks without structurally modifying the dam (focuses on way to address the average annual life loss (AALL) component of the risk equation).

5.1.1. Development of Initial Measures

Three organizational concepts (Figure 5-1) were applied to the development and screening of risk management measures. These concepts include reducing the load on the dam, reducing the probability of a dam breach, and reducing the consequences in the event of a dam breach.

\[ \text{Risk} = f(\text{Hazard, Performance, Consequences}) \]

Figure 5-1: Management measure development concepts
As described in Chapter 4, the 2013 BCRA resulted in three potential failure modes that significantly contribute to the total risk being above tolerable risk guidelines (PFM 33, 34, & 35). Based on this understanding, over ninety measures were formulated to address spillway monolith instability (PFM 33), non-overflow monolith instability (PFM 34), and abutment monolith instability (PFM 35). A list of these initial measures can be found in Table H-1 in Appendix H. Table H-1 of initial measures in Appendix H identifies the failure modes being addressed as well as the rationale for the screening decision. As required by ER-1110-2-1156, a Value Engineering (VE) study was performed during the initial measures screening process. The Value Engineering report can be found in Appendix P.

5.1.2. Development of Initial Risk Management Plans
An initial array of RMPs were formulated using the list of measures in Table H-1 in Appendix H to address the dam safety issues identified in the 2013 BCRA. Between ten and fifteen RMPs were initially developed. The initial array of RMPs were evaluated by completing feasibility level design, rough order of magnitude (ROM) cost estimates, qualitative risk reduction, and qualitative assessment of environmental impacts. The initial array of plans consisted of a new stilling basin (to address PFM 33) with multiple variations of overtopping measures (to address PFM 34 & 35). The details of this initial array of plans can be found in the presentation that was given at the December 2015 RMP meeting and can be found in Appendix H.

For background, two RMP meetings were held during this phase of formulation the first one was in April 2014 and the second in December 2015 (presentations for both meeting can be found in Appendix H). The plans presented at the first RMP meeting (April 2014) were based on the 2013 BCRA results. The plans presented at the second RMP Meeting (Dec 2015) were based on the 2013 BCRA and updated flood hazard. Figure 5-2 provides a timeline of the iterations of formulation that occurred.
Figure 5-2: Plan Formulation Historical Timeline in Relation to Multiple Risk Assessments

(RMMID = Risk Management Measures Identification Meeting)
5.2. Current Formulation Efforts based on 2016 ECRA/FWAC

After the evaluation of the initial array of plans described in section 5.1, a revised risk assessment was completed (referred to as the 2016 ECRA/FWAC) based on the most current updates to the flood hazard curve. In August 2015 the team reinitiated formulation after a directed pause due to the ongoing updates to the flood hazard. The team presented the preliminary results of the 2016 ECRA/FWAC to the Tier 3 vertical team in March 2016, along with the TSP. The team gained concurrence with the ECRA & FWAC risk assessment and TSP. However, the team completed the Final Risk Assessment Technical Summary (ECRA/FWAC) Report in June 2016 which confirmed the dam safety issue(s) for the DSMS (US Army Corps of Engineers, Bluestone Dam Risk Assessment Technical Summary Report, 2016). The 2016 ECRA/FWAC risk assessment resulted in only one failure mode, PFM 33, which contributes to dam safety risk being above tolerable risk guidelines. As a result, the team revised the formulation of measures and plans. This section (5.2) will present the iterations of the plan formulation process that took place to address PFM 33 with the updated flood hazard. Where deemed appropriate overtopping measures were evaluated as an ALARP considerations to address PFM 34 & 35.

In focusing formulation efforts to address PFM 33 with ALARP considerations, previous iterations of formulation indicated that there was only one reasonable RMP that adequately reduced risks associated with PFM 33. As previously stated, in addition to the ECRA/FWAC risk assessments a TSP was selected and endorsed at the 4 March 2016 Mega-Project Tier 3 Decision Point (See Appendix H for Meeting Briefings). The endorsed TSP from the vertical team (LRH, LRD, DSMMCX, RMC, and HQ) is the Stilling Basin with Super-Cavitating Baffles, RMP 6 (Martin, PE, PMP, 2017).

The following section summarizes how measures were combined into plans and evaluated and compared against the FWAC (No Action) plan and then against each other to support the endorsed TSP. In summary, RMP 6 was the only alternative that meets the study objectives of DSMS effectively, efficiently and without causing significant permanent environmental impacts. RMP 6 was identified and vertically endorsed prior to conducting a with-project risk assessment. However, a with-project risk assessment was applied to RMP 6 to ensure it did meet required risk reduction goals and is reported in Chapter 7.

Therefore, the evaluation and comparison is scaled to the appropriate level of effort applied during the formulation process. A review of Appendix H will show that several iterations of formulation were performed when developing risk management measures and plans. For both clarity and brevity, the actual final array of RMPs was renumbered for this reporting to reflect the ultimate and concluding analysis that supports the selected plan (RMP 6).

5.2.1. Iteration 1 – Development & Screening of Risk Management Measures

The risk management measures that were considered for RMP development are presented in Table 5-1.
### Table 5-1: Development of Risk Management Measures for 2016 ECRA/FWAC

<table>
<thead>
<tr>
<th>Measure</th>
<th>Type</th>
<th>Risk Component the Measure Addresses</th>
<th>Retained for Further Consideration</th>
</tr>
</thead>
<tbody>
<tr>
<td>Downstream Conventional Stilling Basin</td>
<td>Structural</td>
<td>Probability of Breach (PFM 33)</td>
<td>Yes</td>
</tr>
<tr>
<td>Stilling Basin with Super-cavitating Baffles</td>
<td>Structural</td>
<td>Probability of Breach (PFM 33)</td>
<td>Yes</td>
</tr>
<tr>
<td>Transitional Flip Stilling Basin</td>
<td>Structural</td>
<td>Probability of Breach (PFM 33)</td>
<td>Yes</td>
</tr>
<tr>
<td>Concrete Overlay of Exposed Rock in Stilling Basin</td>
<td>Structural</td>
<td>Probability of Breach (PFM 33)</td>
<td>Yes</td>
</tr>
<tr>
<td>Stabilize Weir</td>
<td>Structural</td>
<td>Probability of Breach (PFM 33)</td>
<td>Yes</td>
</tr>
<tr>
<td>Parapet Wall</td>
<td>Structural</td>
<td>Probability of Breach (PFMs 34 &amp; 35)</td>
<td>Yes</td>
</tr>
<tr>
<td>Raise the Non-overflow Sections</td>
<td>Structural</td>
<td>Probability of Breach (PFMs 34 &amp; 35)</td>
<td>No</td>
</tr>
<tr>
<td>Remove or Replace Top of Spillway Bridge Beams and Gate Hoisting machinery</td>
<td>Structural</td>
<td>Probability of Breach (PFMs 34 &amp; 35)</td>
<td>Yes</td>
</tr>
<tr>
<td>Flood Proof and Armor Abutments</td>
<td>Structural</td>
<td>Probability of Breach (PFMs 34 &amp; 35)</td>
<td>No</td>
</tr>
<tr>
<td>Install Auxiliary Discharge Features (gated tunnel or uncontrolled saddle spillway through adjacent hillside)</td>
<td>Structural</td>
<td>Probability of Breach (PFMs 34 &amp; 35)</td>
<td>No</td>
</tr>
<tr>
<td>Remove Structure</td>
<td>Structural</td>
<td>Probability of Breach (all PFMs)</td>
<td>Yes</td>
</tr>
<tr>
<td>Replace Structure</td>
<td>Structural</td>
<td>Probability of Breach (all PFMs)</td>
<td>Yes</td>
</tr>
<tr>
<td>Waterproof Dam Openings</td>
<td>Structural</td>
<td>Probability of Breach (PFM 33)</td>
<td>Yes</td>
</tr>
<tr>
<td>Risk Communication</td>
<td>Non-Structural</td>
<td>Consequences</td>
<td>Yes</td>
</tr>
<tr>
<td>Remote Operation of Crest Gates</td>
<td>Non-Structural</td>
<td>Consequences</td>
<td>Yes</td>
</tr>
<tr>
<td>Additional Studies</td>
<td>Non-Structural</td>
<td>Consequences</td>
<td>Yes</td>
</tr>
<tr>
<td>Flood proofing</td>
<td>Non-Structural</td>
<td>Consequences</td>
<td>No</td>
</tr>
<tr>
<td>Flood Warning Systems</td>
<td>Non-Structural</td>
<td>Consequences</td>
<td>No</td>
</tr>
<tr>
<td>Relocation of Structures</td>
<td>Non-Structural</td>
<td>Consequences</td>
<td>No</td>
</tr>
</tbody>
</table>

**5.2.1.1. Screening of Overtopping Measures**

The decision to refocus formulation on addressing PFM 33 and not address the full hydrologic hazard, including any risks associated with overtopping, was made at 4 March 2016 Mega-Project Tier 3 Decision Point (Martin, PE, PMP, 2017) due to the remote probability of occurrence. Therefore, consideration of significant investments to address PFM 34 and PFM 35 were no longer considered warranted to address the goals and objectives of the DSMS. It was determined that means to reduce overtopping risks (PFM 34...
& 35) would only be considered if deemed practicable in the context of ALARP. See Appendix H for the Mega-Project Tier 3 Decision Point Meeting Briefings.

The parapet wall was retained because it is the least cost measure that was originally thought to reduce overtopping risk. Several heights of the wall were evaluated, 8-ft and 14-ft. The construction of the 8-ft parapet wall began under the DSA project and is estimated to cost approximately $3M to $4M for completion. Due to the requirement in ER 1110-2-1156 of evaluating a plan that targets full tolerable risk guidelines using ALARP considerations to include applicable essential USACE guidelines, the 14-ft wall along with the replacement and/or removal of the bridge beam and gate hoisting machinery was also retained.

The other overtopping measures were screened out because they were not efficient in reducing risk in comparison to the 8-ft parapet wall. They reduce risk further below tolerable levels, but for a cost that is in excess of ten times the cost of completing the parapet wall. The 8-ft parapet wall was evaluated in the context of the four ALARP principles and can be found in Table 5-7. The further incremental risk is below tolerable risk guidelines, the weaker the rationale for further risk reduction efforts (US Army Corps of Engineers, Engineering Regulation (ER) 1110-2-1156, Safety of Dams - Policy and Procedures, 2014). Therefore, it becomes difficult to justify additional Federal investment for overtopping measures due to their lack of efficiency.

5.2.1.2. Screening of Non-Structural Measures

The team considered non-structural measures, both physical and non-physical. Some of the measures consisted of flood proofing, critical infrastructure protection, operational changes, warning systems, evacuation plans, risk communication and the like. Many of the measures such as flood proofing and relocation of structures were screened out due to level of protection required and magnitude of cost. Most of the critical infrastructure would be impacted by non-breach consequences and protecting these types of facilities would have minimal impact on reducing incremental risk. Details of these non-structural measures and reasons for not carrying them forward can be found in Appendix H, Planning Appendix.

Generally across the board improvements in advanced warning systems and evacuation plans are considered as part of the FWAC assumptions. The majority of life loss is estimated to occur in and around the Charleston area, and their emergency warning and evacuation planning is continually improving with increased planning and additional warning capabilities. For this reason, substantial improvements to the estimated existing condition mobilization curves are already included in the FWAC assumptions. For these types of non-structural measures to be improved above and beyond what is assumed in the FWAC condition, may not be feasible and could not be estimated with confidence. Additionally, it is estimated that life loss from breach would need to be reduced by a full order of magnitude or more to achieve tolerable risk in the absence of a structural measure to address the system response breach probability. Due to the large amount of population at risk and the extreme flood depths at higher breach levels, no amount of improved emergency planning or response is likely to confidently reduce potential life loss by an order of magnitude, particularly since those improvements would need to be above and beyond the improvements that are expected to occur in the FWAC.

Three nonstructural measures were carried forward; improved risk communication, remote operations of spillway crest gates, and additional studies. Improved risk communication is carried forward as a nonstructural measure since it was the most effective way of reducing risk through improved messaging and understanding. Through this continued risk communication, it will serve as a reminder for state and local officials to continue to carry out plans for improving warning and evacuation systems and reduce the uncertainty about whether those improvements will actually occur in the FWAC. Risk communication will
also assist the emergency managers in understanding what the risks are and how they can develop their evacuation and response plans to maximize their effectiveness. Risk communication could include regular outreach meetings with emergency managers and operators of critical infrastructure and industrial facilities (especially the major chemical industry facilities), the sharing of resources such as inundation and arrival mapping and technical expertise, and the continual maintenance of relationships and contacts between local emergency managers and USACE personnel. A detailed scope of risk communication activities would be developed during the PED phase.

5.2.2. Iteration 1 – Development and Screening of Risk Management Plans

Plans were developed by combining the retained measures from iteration 1 into RMPs. The plans were formulated with the intent of meeting the study objectives and taking advantage of the opportunities. The RMPs include the five required alternatives specified in Engineering Regulation (ER) 1110-2-1156. The five required alternatives are: No action (FWAC) alternative; meeting full tolerable risk guidelines using ALARP considering to include applicable essential USACE guidelines; achieving only tolerable risk limit for life-safety; remove structure; and replace structure. The plans developed during iteration 1 are in the list below as well as in Table 5-2.

- No Action (RMP 1)
- Dam Removal (RMP2)
- Dam Replacement (RMP 3)
- Meeting full tolerable risk guidelines using ALARP considerations
  - Downstream Conventional Stilling Basin (RMP 4)
  - Transitional Flip Stilling Basin (RMP 5)
  - Stilling Basin with Super-cavitating Baffles with an 8-foot parapet wall (RMP 6)
- Achieving only tolerable risk limit for life-safety
  - Concrete Overlay in Stilling Basin (RMP 7)
- Meeting full tolerable risk guidelines using ALARP considerations to include applicable essential USACE guidelines
  - Stilling Basin with Super-cavitating Baffles; raised crest gates and bridge; 14-foot parapet wall; and additional anchors in dam required for stability at full IDF. (RMP 8)

### Table 5-2: Risk Management Plans and Associated Measures

<table>
<thead>
<tr>
<th>Plans</th>
<th>Measures Included</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Downstream</td>
</tr>
<tr>
<td>2</td>
<td>Conventional</td>
</tr>
<tr>
<td>3</td>
<td>Stilling Basin</td>
</tr>
<tr>
<td>4</td>
<td>Transient</td>
</tr>
<tr>
<td>5</td>
<td>Stilling Basin</td>
</tr>
<tr>
<td>6</td>
<td>Transient</td>
</tr>
<tr>
<td>7</td>
<td>Stilling Basin</td>
</tr>
<tr>
<td>8</td>
<td>Transient</td>
</tr>
</tbody>
</table>

- Downstream Conventional Stilling Basin
- Transient Flip Stilling Basin
- Stilling Basin with Super-cavitating Baffles
- Concrete Overlay
- Stabilize Weir
- Parapet Wall
- Remove Dam
- Replace Dam
- Risk Communication
- Waterproof Dam
- Raise Bridge Beam and Gate Machinery

129 June 2017
5.2.3. Description of Risk Management Plans

5.2.3.1. RMP 1 - FWAC (No Action)
The FWAC (No Action) Plan is described in Section 3.2.3; for brevity it will not be repeated here.

Note: The FWAC assumes the successful completion through Phase 4 of the current DSA construction project, installation of additional anchors (66 +/-), improved warning and evacuation systems, and improved public education (risk communication). As the additional anchors are required to meet the existing risks portrayed, these anchors will have to be installed in the FWAC (No Action) Plan. It should be noted, these anchors are also included in many of the RMPs discussed in this chapter.

As stated in Section 3.2.3.8 and 3.2.3.9, improved warning systems and evacuations plans will be implemented by the local entities and the study assumes these will be implemented whether the recommended plan is constructed or not. Additional investments to improved warning systems above and beyond what the locals are anticipated to implement was determined to provide negligible risk reduction; therefore, additional improvements were not considered in development of alternatives.

5.2.3.2. RMP 2 - Dam Removal
This plan includes removal of all or a portion of the Bluestone Dam to eliminate the impoundment. Flows would return to pre-dam conditions eliminating the ability to meet its originally authorized purpose. Flood risk management benefits, which are estimated at greater than $87M annually (FY15 dollars), would no longer be realized. It is expected that the sediment deposited within the reservoir area would be released in whole or in part causing significant impact to downstream Resource Category 1 habitat (recently renamed as “High Value Habitat”, but will be referred to as “Resource Category 1” in this document). Mitigation measures to minimize these effects are part of this plan. This plan also includes improved public education regarding the risk of flooding due to the loss of this dam. The improved educational tools will be developed and implemented in cooperation with local stakeholders and government bodies, as well as state and federal emergency management agencies.

Mitigation requirements as determined the Supplemental Final Environmental Impact Statement (SFEIS) will be part of this plan.

5.2.3.3. RMP 3 - Dam Replacement
This alternative consists of removing the existing dam and designing a new dam to meet all applicable essential USACE guidelines. The replacement dam would be a concrete structure with a spillway, similar to the original design. The replacement would be expected to address all failure modes and meet all tolerable risk guidelines. This plan would also include any mitigation required for impacts to the environment. This plan also includes the development of improved public education regarding the risk of flooding during construction and following construction as even the new dam is unlikely to eliminate all flood hazards. The improved educational tools will be developed and implemented in cooperation with local stakeholders and government bodies, as well as state and federal emergency management agencies.

Mitigation requirements as determined the SFEIS will be part of this plan.

5.2.3.4. RMP 4 - Downstream Conventional Stilling Basin
This plan does not address floods that overtop the dam (Inflow Design Flood (IDF)). This plan includes the construction of a new stilling basin with baffles downstream of the existing stilling basin (For details see Figure 5-6 through Figure 5-8). This new stilling basin extends approximately 300 feet downstream of the end sill of the existing stilling weir apron to the downstream end of the new basin. The existing baffles and endsill of the spillway apron are removed with this plan. The existing weir and weir apron are also
removed and replaced with a transition zone between the toe of dam at EL 1,368-ft and the new, deeper stilling basin at EL 1,345-ft, requiring large quantities of excavation of rock. The entire length of the channel chute and transition zone are covered with concrete, and baffle blocks are constructed near the end of the new basin along with appropriate uplift relief drainage features. This plan also consists of a new stilling basin wall constructed to EL 1,425-ft along the right and left bank of the new stilling basin.

This plan also includes the development of improved public education regarding the risk of living downstream of a dam. The development and implementation of these tools will be in cooperation with local stakeholders and government bodies, as well as state and federal emergency management agencies. Since this plan is not designed for floods that overtop the dam, waterproofing of openings are part of this plan to prevent flooding of the drainage galleries.

In addition, the approximate 66 anchors of the FWAC measures that were approved in the 1998 DSA report, but never procured, will be installed. Construction of these anchors was included in the ECRA and FWAC risk assessments and will be implemented with construction general (CG) funds. The anchors will be designed and procured with the dam safety modification (DSM) project to ensure compatibility with the proposed design. The remote operation of the crest gates will be implemented due to the risk reduction gained for life safety risk to dam personnel.

Mitigation requirements as determined the SFEIS will be part of this plan.

**5.2.3.5. RMP 5 - Transitional Flip Basin**

This plan does not address floods that overtop the dam (IDF). This alternative plan would include the construction of a new concrete apron slab within the existing stilling basin with foundation anchors and appropriate uplift relief drainage features that would transition to a flip bucket spillway just upstream of the existing weir (See Figure 5-9 through Figure 5-11). Discharges up to a certain threshold, approximately half the design discharge, will result in a hydraulic jump upstream of the flip bucket due to the sloping apron. Greater discharges will result in the formation of a flip that will transmit discharge downstream of the basin. Often a flip basin spillway would be constructed with an adjacent downstream plunge pool of adequate depth to allow the water to fall into downstream waters without creating a large scour hole. If plunge pool construction were included in this alternative plan, such a pool would impact additional Resource Category 1 habitat. If this plan does not include an adjacent plunge pool, it would include the construction of a concrete cutoff wall downstream of the existing stilling weir apron to prevent possible scour caused by the plunging water from migrating upstream and compromising the stability of the dam. Since RMP 5 is not designed for floods that overtop the dam, waterproofing of openings is included in this plan to prevent flooding of the drainage galleries.

This plan also includes the development of improved public education regarding the risk of living downstream of a dam. This would be developed and implemented in cooperation with local stakeholders and government bodies, as well as state and Federal emergency management agencies.

Mitigation requirements as determined the SFEIS will be part of this plan.

In addition, the stilling weir will be stabilized and the approximately 66 anchors will be installed. The anchors are part of the FWAC condition, but will be implemented with construction general (CG) funds because they are required to meet the estimated ECRA and FWAC. The anchors will be designed and procured with the dam safety modification project to ensure compatibility. The remote operation of the crest gates will be implemented due to the risk reduction gained for life safety risk to dam personnel.
5.2.3.6. RMP 6 - Stilling Basin with Super-cavitating Baffles

This risk management plan consist of various features and risk management measures formulated to ensure stability of the stilling basin and the dam during extreme flood events. Under this plan, the modified stilling basin would remain a two stage system within the same footprint with the following modifications and features (see Figure 5-12 through Figure 5-15 for additional details):

- A protective concrete apron overlay for the 182 feet of natural riverbed in the first stage between the dam and the existing stilling weir with an system of underdrains
- Construction of a permanent divider wall to bisect stilling basin with a new gallery plumbed to the underdrains in the apron
- Demolition of the existing first stage baffle blocks and construction of new, larger super-cavitating baffle blocks
- Anchors in both the existing and new concrete slabs (apron) to stabilize against uplift pressures in the foundation created by underseepage from the reservoir
- Construction of a new drainage gallery within the dam to relieve some of the uplift pressures
- Stabilizing the left and right training walls with anchors
- Placement of scour protection behind the left and right training walls
- Installation of stabilization anchors in the stilling weir
- Demolition/reconstruction of the second stage end sill and baffle blocks within their existing footprint and anchoring to ensure stability and satisfactory performance
- The completion of the 8-ft tall parapet wall in the non-overlow section to reduce the probability of overtopping.

RMP 6 does not meet all essential USACE guidelines because it does not address the full IDF. Therefore, waterproofing of openings is included in this plan to prevent flooding of the drainage galleries.

This plan also includes the development of improved public education regarding the risk of flooding due to a dam failure. This would be developed and implemented in cooperation with local stakeholders and government bodies, as well as state and Federal emergency management agencies.

In addition, this plan would include approximately 66 anchors which are part of the FWAC condition. The anchors will be implemented using CG funds because they are approved under the DSA project, but were never procured. The anchors are required to meet the estimated risk in the ECRA and FWAC. The anchors will be designed and procured with the dam safety modification project to ensure compatibility.

5.2.3.7. RMP 7 - Concrete Overlay of Exposed Rock in Stilling Basin

This plan targets achieving only the tolerable risk limit for life safety only. This plan is very similar to RMP 6 except it doesn’t include the super-cavitating baffles or the training wall improvements. This alternative includes a protective concrete apron overlay for the approximately 180 feet of natural riverbed in the first stage between the apron and the existing stilling weir. To further stabilize the foundation against pressure created by seepage from the reservoir pool, this alternative would include anchors in both the existing apron and new concrete slabs (See Figure 5-16 and Figure 5-17). This plan was developed to reduce the risk just below TRG and was not intended to address all the risk. This plan is very similar to RMP 6, except that it does not address the hydraulic performance of the basin, therefore does not include the super baffles and training wall improvements. This plan would likely include a divider wall, but the details of the drainage for uplift relieve would be less than that of RMP 6. Remote operation of the 21 crest gates would also be part of this plan.
This plan also includes the development of improved public education regarding the risk of living downstream of a dam. This will be developed and implemented in cooperation with local stakeholders and government bodies, as well as state and Federal emergency management agencies.

Mitigation requirements as determined the SFEIS will be part of this plan. In addition, the stilling weir will be stabilized and the approximate 66 anchors will be installed. The anchors are part of the FWAC condition, but will be implemented with CG funds because they are required to meet the estimated ECRA. The anchors will be designed and procured with the dam safety modification project to ensure compatibility. The remote operation of the crest gates will be implemented due to the risk reduction gained for life safety risk to dam personnel.

5.2.3.8. RMP 8 - Stilling Basin with Super-cavitating Baffles (14-ft Parapet Wall)

This risk management plan consists of various features and risk management measures formulated to ensure stability of the stilling basin and the dam during extreme flood events. RMP 8 was formulated with the intention of achieving all essential USACE guidelines and is designed to safely pass the full IDF. This plan consist of every measure included in RMP 6 except for the remote operation of the spillway crest gates. This plan consists of a modified stilling basin with super-cavitating baffles; raised crest gates and bridge; higher (14-foot) parapet wall; and additional anchors in dam required for stability at the IDF. There are a few additional measures above and beyond RMP 6 that are included part of RMP 8 and are listed in Table 5-3.

This plan also includes the development of improved public education regarding the risk of living downstream of a dam. This would be developed and implemented in cooperation with local stakeholders and government bodies, as well as state and federal emergency management agencies.

Mitigation requirements as determined the SFEIS will be part of this plan.

In addition, this plan would include approximately 66 anchors which are part of the FWAC condition. The anchors will be implemented using CG funds because they are approved under the DSA project, but were never procured. The anchors are required to meet the estimated risk in the ECRA and FWAC. The anchors will be designed and procured with the dam safety modification project to ensure compatibility.

**Table 5-3: RMP 8 Additional Measures Above and Beyond RMP 6**

<table>
<thead>
<tr>
<th>Additional Measures</th>
<th>Description</th>
<th>Purpose</th>
</tr>
</thead>
<tbody>
<tr>
<td>Raise Bridge Beams and Crest Gates</td>
<td>Demolition gantry crane bridge and reconstruct at a higher elevation and reconfigure gate operation equipment to ensure unobstructed spillway flow up to the IDF.</td>
<td>Current configuration creates orifice flow conditions. Removing these obstructions results in more efficient spillway flow reducing the PMF reservoir elevation from EL 1,553-ft to EL 1,547-ft</td>
</tr>
<tr>
<td>Parapet Wall</td>
<td>Minimum of 14-ft additional dam height</td>
<td>To safely pass the IDF with minimum required freeboard.</td>
</tr>
<tr>
<td>Additional Anchors</td>
<td>Approximately two additional anchors per monolith (above and beyond the approximate 66 anchors required to meet risk estimated in ECRA).</td>
<td>The original anchor design is based on a design EL 1,542-ft, additional anchors will be required to meet the minimum factor of safety for the higher PMF elevation.</td>
</tr>
<tr>
<td>More Robust Stilling Basin Features</td>
<td>See Table 6-1 for description details.</td>
<td>To meet the minimum required factors of safety for the IDF.</td>
</tr>
</tbody>
</table>
5.2.4. Evaluation Criteria for Screening RMPs
Each plan was formulated in consideration of completeness, efficiency, effectiveness, and acceptability as described in the Planning Guidance Notebook (ER 1105-2-100) and were also used as the primary evaluation criteria to compare and screen plans. Two additional criteria were also used, engineering (technical) feasibility and environmental effects, to further distinguish between plans. Each of the criteria are defined below.

- Effectiveness (Incremental Risk Reduction): is the extent to which a RMP contributes to achieving the planning objectives of reducing incremental risk.
- Efficiency: is the extent to which a RMP is the most cost effective means of achieving the objectives.
- Acceptability: is the extent to which a RMP is acceptable in terms of applicable laws, regulations and public policies.
- Completeness: is the extent to which a RMP provides and accounts for all necessary investments or other actions to ensure the realization of the DSMS risk management objectives, including actions by other Federal and non-Federal entities.
- Engineering (Technically) Feasibility: used to consider whether a RMP can reasonably be constructed, including any significant engineering or construction constraints. Consideration was given to things such as technical feasibility, and if a plan creates risk to the dam during or after construction.
- Environmental Effects: takes into account the effects of the RMP on the human environment including ecological, cultural and social effects.

5.2.5. Iteration 1 – Evaluation of Eliminated Risk Management Plans
Each RMP was evaluated in consideration of the criteria as previously described in Section 5.2.4. Three plans were eliminated from further consideration during Iteration 1. The plans that were eliminated are RMP 2 (Dam Removal), RMP 3 (Dam Replacement) and RMP 8 (Modified Stilling Basin designed to pass the full IDF). An evaluation summary of the plans that were eliminated in iteration 1 can be found in Table 5-4.

5.2.5.1. RMP 2 - Summary of Evaluation
This plan would completely eliminate the risk associated with dam failure (incremental risk) because there would no longer be a structure to fail. The downstream communities would not receive the original authorized benefits for flood risk management. Flood risk management benefits are estimated to be more than $87M annually. A cursory cost estimate was completed and is expect to exceed $200M for removing the structure. Environmental mitigation costs were not considered in the cursory level cost estimate, but are expected to be high due to disposal of material, transportation, and demolition activities that would have long term effects on communities and the natural environment.

This alternative is only acceptable if the federal interest for the project is rescinded. It would require deauthorization of the authorized project purposes. The dam provides public health and safety to the downstream communities and as previously stated, provides $87 Million in average annual flood risk
management benefits. Removing the dam would cause flooding more frequently to the downstream communities which undermines the authorized purpose of flood risk management. The authorized benefits for hydropower and recreation would also not be realized under this alternative.

The technical details for removing the dam were not fully developed; however, it is believed that decommissioning and removing the dam would be challenging and complex. The dam consists of 1.3M cubic yards of concrete and the demolition and disposal alone would likely take 5 to 10 years. Finding an area for this amount of material could be difficult and may result in significant terrestrial impacts depending on the location of the disposal site. It is expected sediment, which has deposited within the reservoir area since the original dam construction, would be released in whole or in part causing short term impacts to the downstream reaches including the high quality Resource Category 1 habitat. The long term benefits to dam removal may result in increased aquatic habitat due to the area being reverted back to a more natural riverine system. The science of monitoring is relatively new concerning the long term success and the detriments of dam removal outcomes. The effort that it would take to remove Bluestone Dam would result in significant short term impacts to the human environment due to the deconstruction effort necessary to remove the large structure and associated infrastructure. Dam removal efforts also is expected to result in short term impacts to both terrestrial and aquatic environment within the footprint of the work limits. Negative impacts on the human environment and on the surrounding communities due to the loss of recreational impoundment behind the dam is also expected with implementation of this plan.

In summary, this plan was eliminated from further consideration primarily because it would eliminate authorized project benefits. Floodplain development has adjusted to the dam being in place and removal of the dam would result in more flood damages more frequently. In other words, this plan would increase non-breach flood risk which does not meet the objective of the study and was eliminated from further consideration.

5.2.5.2. RMP 3 - Summary of Evaluation

This alternative includes removing the existing dam structure and replacing it with a new structure that would be designed to address all dam safety risks (failure modes) and all applicable essential USACE guidelines. This plan would result in Bluestone Dam being a DSAC 5 structure and the risk would be far below tolerable risk guidelines for both annual probability of failure (APF) and average annual life loss (AALL). This plan is inefficient when compared to other alternatives that also achieve risk management objectives. A full cost estimate was not developed but a cursory analysis indicated cost will likely exceed $1B. It is expected this plan will have the longest construction schedule meaning in the exposure to intolerable risks would be the highest.

Construction of RMP 3 would likely result in significant permanent impacts to Resource category 1 habitat. The U.S. Fish and wildlife service mitigation policy, established in accordance with the Fish and Wildlife Coordination Act (16 U.S.C 661-667(e)) and the National Environmental Policy Act (42 U.S.C. 4321-4347) defined resource categories to assist in determining appropriate mitigation goals for federal water resource development projects. According to the policy, the designation criteria for habitat in Resource Category 1 is "habitat to be impacted is of high value for evaluation species and is unique and irreplaceable on a national basis or in the ecoregion section." The mitigation goal for habitat in Resource Category 1 is "no loss of existing habitat value." As there are other reasonable alternatives available which would meet objectives and avoid permanent impact to this high value habitat, this alternative is considered unacceptable in terms of applicable laws, regulations and public policies.

This plan is complete as it would provide and accounts for all necessary investments or other actions to ensure the realization of the DSMS risk management objectives. The technical details for this plan were
not fully developed. Though construction of a new dam was determined feasible, it was recognized there would be substantial challenges in locating a site which is as efficient as the existing to capture flood waters. Complexities in construction of a new dam and removal of the existing dam were also recognized.

Regardless of location, significant adverse impacts would result to the environment, but the severity of the impacts would be dependent on which location was chosen as the new dam site. If the new dam site was chosen downstream of the existing dam location, significant and permanent impacts to a known Resource Category 1 aquatic habitat would occur along with terrestrial resources. Significant impacts to the human environment and communities downstream would also occur as a result of the new dam construction. If the dam replacement site was chosen upstream of the existing dam, partial flood capacity and recreational lake use would be lost. Flood control of tributaries that flow into the lake currently, including the Bluestone River, could also be lost resulting in higher incident of flooding downstream. Regardless of the location chosen for a new dam, significant adverse environmental impacts would be actualized as the result of constructing a new dam and removing the current dam. All impacts associated with dam removal would also be valid for dam replacement.

The plan is clearly inefficient when compared to other alternatives which achieve risk management objectives (tolerable risk guidelines). There are also concerns regarding complexity and technical feasibility of removing the existing dam and finding a site location as efficient as the existing one to capture flood waters. Moreover, there would be permanent adverse effect to Resource Category 1 habitat, or other sensitive habitat associated with the construction of a new dam and removal of the existing dam. As other more reasonable alternatives exist to meeting the study objectives, further consideration of RMP 3 is eliminated.

5.2.5.3. RMP 8 – Summary of Evaluation

Although this plan meets the risk reduction objectives, due to high cost and long construction schedule, it is determined to be inefficient when compared to other RMPs. After implementation of RMP 6 (TSP), the risk is low (APF = 8.52E-07 and AALL = 5.06E-04). When reducing risk even further with implementation of RMP 8, it becomes inefficient due to the high cost. RMP 8 is expected to cost $120M to $150M greater than the cost of RMP 6 for little risk reduction.

Environmental impacts with implementation of RMP 8 are marginally greater than that of RMP 6 due to additional impacts from disposal material. Aquatic impacts are expected to be similar to that of RMP 6. Due to the inefficiency of RMP 8 it was eliminated and not carried forward for further consideration.
<table>
<thead>
<tr>
<th>Evaluation &amp; Summary Criteria</th>
<th>RMP 1 FWAC (No Action)</th>
<th>RMP 2 Remove Dam</th>
<th>RMP 3 Replace Dam</th>
<th>RMP 8 Modified Stilling Basin Full PMF</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Effectiveness</strong> (Incremental Risk Reduction)</td>
<td>Not effective. There will be minimal risk reduction beyond DSA construction. Risks would remain above tolerable risk guidelines.</td>
<td>Effective at reducing incremental risk. However, would significantly increase non-breach risk and would eliminate authorized purposes of flood risk management</td>
<td>Effective at reducing risk for breach risks.</td>
<td>Effective. Expected to reduce risk below tolerable risk guidelines</td>
</tr>
<tr>
<td><strong>Efficiency (Cost)</strong></td>
<td>~$15M to install 66 dam anchors to meet risk condition estimated in 2016 ECRA</td>
<td>A full cost estimate was not developed, however cursory estimates of construction cost exceed $200M. Significant environmental effects leading to high mitigation cost.</td>
<td>A full cost estimate was not developed, however cursory estimates of construction cost exceed $1B. This is expected to be the most expensive plan.</td>
<td>This plan is expected to be above $500M.</td>
</tr>
<tr>
<td><strong>Acceptability</strong></td>
<td>Not acceptable because risks would not be within the agency’s tolerable risk guidelines.</td>
<td>Acceptable. However, it would increase non-breach risk for more frequent flood events which eliminates authorized purposes of flood risk management.</td>
<td>Not acceptable. Environmental impacts and potential consequences to a Resource Category 1 habitat would be significant.</td>
<td>Acceptable; however, it has aesthetics issues because of the height of the parapet wall (14-ft)</td>
</tr>
<tr>
<td><strong>Completeness</strong></td>
<td>Not Complete</td>
<td>Complete</td>
<td>Complete</td>
<td>Complete</td>
</tr>
<tr>
<td><strong>Engineering (Technically Feasible)</strong></td>
<td>It is constructible. A work platform will have to be installed across the primary spillway</td>
<td>It is constructible. However, release of accumulated sediment may create complexities in implementation.</td>
<td>There are constructability issues and complexities associated with siting and constructing new dam.</td>
<td>It is constructible.</td>
</tr>
<tr>
<td><strong>Environmental Impacts</strong></td>
<td>Minimal from installation of dam anchors. However, if breach were to occur, impacts would be significant and permanent</td>
<td>Short term adverse effects associated with demolition, spoil, release of sediment. Long-term beneficial effects associated with return to free flowing stream.</td>
<td>Significant permanent impacts due to construction likely within a Resource Category 1 habitat.</td>
<td>Permanent features are within the existing dam footprint. Temporary Impact of Resource Category 1 habitat</td>
</tr>
<tr>
<td><strong>Conclusion of Screening process</strong></td>
<td>RETAINED for comparison purposes</td>
<td>Eliminated. Elimination of flood risk management purpose and increase in non-breath risks.</td>
<td>Eliminated. Inefficient compared to other plans. Effects to Resource Category 1 habitat would be significant and permanent.</td>
<td>Eliminated. Due to high cost and long construction schedule (not efficient)</td>
</tr>
</tbody>
</table>
5.2.6. Iteration 2 – Evaluation and Eliminated Risk Management Plans

This section describes the evaluation and screening process that took place on the remaining RMPs, also referred to as iteration 2. Each of the remaining RMPs was evaluated in consideration of the criteria as previously described in Section 5.2.4. Five plans were carried forward after the first iteration of the planning process based on the 2016 ECRA/FWAC risk assessment. The plans that were carried forwards consist of the following:

- RMP 1 (No Action/FWAC);
- RMP 4 (Downstream Conventional Stilling Basin);
- RMP 5 (Transitional Flip Stilling basin);
- RMP 6 (Modified Stilling Basin with Super-cavitating Baffles and 8-ft Parapet Wall); and
- RMP 7 (Modified Stilling Basin with Concrete Overlay).

The following sections in 5.2.6 describe the evaluation and comparison of each RMP and the selection of the TSP.

5.2.6.1. Evaluation of RMP 1 - FWAC (No Action)

5.2.6.1.1. Effectiveness Evaluation

The No Action Alternative does not provide a long-term solution to reduce the risk of failure from spillway monolith instability. This plan does not reduce risk to below TRGs, but implementation of the additional dam anchors is required to meet the risk condition in the 2016 ECRA.

5.2.6.1.2. Efficiency Evaluation

Installation of additional dam anchors would cost in excess $15M for this plan because a contractor would have to mobilize and install a platform across the primary spillway just to install many of these anchors. The installation of these anchors when compared to the final array of plans is less efficient because the contractor would have to install them as a single component.

5.2.6.1.3. Acceptability Evaluation

Not acceptable to policy because risks would not be within the agency's tolerable risk guidelines.

5.2.6.1.4. Completeness Evaluation

Not complete because it does not meet the objectives of the DSMS.

5.2.6.1.5. Engineering (Technical) Feasibility Evaluation

Installation of the 66 +/- additional anchors will require re-installation of a work platform across the primary spillway.

5.2.6.1.6. Environmental Evaluation

Minimal or no impacts are anticipated by installation of additional dam anchors. However, without additional investments to lower life-safety risks from Bluestone Dam, the dam would potentially breach leaving the safety of the surrounding human and natural environment severely impacted. There would also be subsequent effects to the local and regional economies. Some of the environmental impacts that could occur as a result of Bluestone Dam breaching include significant hazardous contamination, soil displacement, sedimentation, scouring, habitat destruction including wetlands, significant reduction in species abundance and diversity, flora and fauna mortality, impacts to both up and downstream recreational facilities and fishing quality, water quality impacts, significant downstream impacts to cultural resources, and significant risk to public safety downstream of the dam.
5.2.6.1.7. Conclusion for RMP 1 Evaluation
The FWAC (No Action) plan was eliminated as it did not meet the goals and objectives of the DSMS.

5.2.6.2. Evaluation of RMP 4 - Downstream Conventional Stilling Basin

5.2.6.2.1. Effectiveness Evaluation
This alternative is expected to reduce risk to meet tolerable risk guidelines; however, a quantitative risk assessment was not completed for this alternative. It was screened out due to other reasons, such as inefficiency, prior to having a quantitative risk assessment complete. The risk reduction was qualitatively determined to have similar risk reduction to the modified stilling basin with the super-cavitating baffles (RMP 6) because physical hydraulic modeling of both basins showed adequate hydraulic performance. This plan would bring total risk below the tolerable risk threshold, but for a higher construction cost when compared to RMP 6 (TSP).

5.2.6.2.2. Efficiency Evaluation
Rough order of magnitude (ROM) costs were developed to generate a cost range for the downstream conventional stilling basin plan. The range of cost for this alternative is expected to be between $400M and $700M. In comparison to RMP 6, this plan is thought to be less efficient due to the higher ROM cost estimate.

The primary quantities for the ROM costs were developed for the Downstream Conventional Stilling Basin Plan and can be found in Table 5-5.

Table 5-5: Major Quantities for Downstream Conventional Stilling Basin (RMP 4)

<table>
<thead>
<tr>
<th>Material</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>230,000 CY</td>
</tr>
<tr>
<td>Reinforcing Steel</td>
<td>30,000,000 LBS</td>
</tr>
<tr>
<td>Steel Sheet Piling Cofferdam Cells</td>
<td>210,000 SF</td>
</tr>
<tr>
<td>Cofferdam Aggregate Fill</td>
<td>70,000 SF</td>
</tr>
<tr>
<td>Rock Excavation</td>
<td>285,000 CY</td>
</tr>
<tr>
<td>Common Excavation</td>
<td>90,000 CY</td>
</tr>
<tr>
<td>Strand Anchors</td>
<td>4,873 EACH</td>
</tr>
</tbody>
</table>

5.2.6.2.3. Acceptability Evaluation
Construction of RMP 4 would likely result in significant permanent impacts to Resource Category 1 habitat. The U.S. Fish and Wildlife Service mitigation policy, established in accordance with the Fish and Wildlife Coordination Act (16 U.S.C 661-667(e)) and the National Environmental Policy Act (42 U.S.C. 4321-4347) defined resource categories to assist in determining appropriate mitigation goals for federal water resource development projects. According to the policy, the designation criteria for habitat in Resource Category 1 is "habitat to be impacted is of high value for evaluation species and is unique and irreplaceable on a national basis or in the ecoregion section." The mitigation goal for habitat in Resource Category 1 is "no loss of existing habitat value."

As there are other reasonable alternatives available which would meet objectives and avoid permanent impact to this high value habitat, this alternative is difficult to consider in terms of applicable laws, regulations and public policies because the impacts are permanent when there are other plans that are not.
5.2.6.2.4. Completeness Evaluation
This plan is considered complete as it provides and accounts for all necessary investments to ensure the realization of the DSMS risk management objectives.

5.2.6.2.5. Engineering (Technical) Feasibility Evaluation
The downstream conventional stilling basin design was developed using standard design approaches for baffled basins. The design eliminates the existing two stage hydraulic jump basin and provides for a single hydraulic jump with adequate tailwater provided by the downstream channel. The design was proved effective utilizing a 1:36 scale sectional physical model; which represents a width of three spillway bays. The design performed well under the conditions evaluated.

Some of the technical challenges that were not addressed in the feasibility level evaluation were associated with deep excavation for the construction of the stilling basin and the demolition and construction of the chute that would replace the existing two stage hydraulic jump basin. These considerations would add complexity and risk during construction as compared to the transitional flip and the super-baffle basins.

Nevertheless, the stilling basin represents a standard hydraulic jump stilling basin design and is expected to prove effective in meeting the goals of the study.

5.2.6.2.6. Environmental Evaluation
The footprint of this alternative would impact more of the downstream environmental resources than the other alternatives, approximately an additional 300 feet of Resource Category 1 habitat in the New River downstream of the dam. Construction of RMP 4 would result in significantly greater impacts to the Resource category 1 habitat. The U.S. Fish and wildlife service mitigation policy, established in accordance with the Fish and Wildlife Coordination Act (16 U.S.C 661-667(e)) and the National Environmental Policy Act (42 U.S.C. 4321-4347) defined resource categories to assist in determining appropriate mitigation goals for federal water resource development projects. According to the policy, the designation criteria for habitat in Resource Category 1 is "habitat to be impacted is of high value for evaluation species and is unique and irreplaceable on a national basis or in the ecoregion section." The mitigation goal for habitat in Resource Category 1 is "no loss of existing habitat value." As there are other reasonable alternatives available which would meet objectives and avoid permanent impact to this high value habitat, this alternative is considered unacceptable in terms of applicable laws, regulations and public policies.

5.2.6.2.7. Conclusion for RMP 4 Evaluation
This plan appears inefficient when compared to other alternatives which achieve risk management objectives (tolerable risk guidelines). Also, there are concerns with risk during construction when compared to RMP 6 and the complexity of extending the stilling basin downstream. The primary reason this plan is screened out is due to the permanent adverse effect to Resource Category 1 habitat, or other sensitive habitat associated with the construction of this stilling basin that are avoided by other more reasonable alternatives.

5.2.6.3. Evaluation of RMP 5 - Transitional Flip Basin
5.2.6.3.1. Effectiveness Evaluation
The greatest uncertainty relates to the formation of the scour hole downstream of the stilling basin. The scour hole downstream of the stilling basin gradually increased in depth when RMP 6 was tested. This was not the case with the Transitional Flip alternative. The depth of the scour hole increased suddenly and
dramatically when the hydraulic jump was swept out of the stilling basin. Such sudden and dramatic increase in scour depth signifies loss of control; unless the scour counter measures such as cutoff wall or a preformed scour hole downstream of the flip are incorporated (Stantec Consulting Services, 2015).

Such loss of control does not necessarily imply poor engineering, provided that the consequences can be predicted with a measure of certainty and can be minimized within acceptable limits. These risks were not quantified, but are, from a qualitative point of view, considered greater than the risks associated with the other alternatives (Stantec Consulting Services, 2015).

5.2.6.3.2. Efficiency Evaluation
A screening level cost estimate was not developed for this plan as the uncertainty concerning its effectiveness was considered sufficient to screen out this alternative. However, given the scope of this project would involve similar construction features as the RMP 6, including concrete scour protection, additional rock and uplift anchors and relief features, it is likely this alternative would be similar in cost. A significant cost increase could occur if a preformed scour hole or cutoff wall would be required.

5.2.6.3.3. Acceptability Evaluation
Construction of RMP 5 would likely result in significant permanent impacts to Resource Category 1 habitat if a preformed scour hole or cutoff wall is constructed. The U.S. Fish and wildlife service mitigation policy, established in accordance with the Fish and Wildlife Coordination Act (16 U.S.C 661-667(e)) and the National Environmental Policy Act (42 U.S.C. 4321-4347) defined resource categories to assist in determining appropriate mitigation goals for federal water resource development projects. According to the policy, the designation criteria for habitat in Resource Category 1 is "habitat to be impacted is of high value for evaluation species and is unique and irreplaceable on a national basis or in the ecoregion section." The mitigation goal for habitat in Resource Category 1 is "no loss of existing habitat value."

As there are other reasonable alternatives available which would meet objectives and avoid permanent impact to this high value habitat, this alternative is considered unacceptable in terms of applicable laws, regulations and public policies.

5.2.6.3.4. Completeness Evaluation
This plan is considered complete as it provides and accounts for all necessary investments to ensure the realization of the DSMS risk management objectives.

5.2.6.3.5. Engineering (Technical) Feasibility Evaluation
The transitional hydraulic jump basin consists of removing the existing 1st stage baffles, paving exposed rock section of the 1st stage, and constructing a flip structure along the upstream face of the existing stilling weir. The concept is that the basin would function as a hydraulic jump basin up to the original design discharge, then transition into a flip basin for discharges above the original design. Various combinations of flip trajectory and dentates were evaluated.

A team of experts were consulted to evaluate the scour and hydraulics of various flip basin alternatives. Experts observed physical models of risk management plan structural alternatives and provided professional opinions on the viability of each alternative. The experts concluded that “none of the flip alternatives are viable” (Stantec Consulting Services, 2015).

The concept for a flip is to project the water downstream of the secondary basin and thus prevent erosion or undermining of the structure. The concept behind the dentated flip is to decrease the stream power per unit area at the impact point because the impact will occur over a larger area. In
order for the flips to function, the tailwater must be at or below the lip of the flip. If the lip is placed above the high tailwater observed at Bluestone, the water level in the primary basin would submerge the sluices prior to sweepout of the basin, which is not desirable. All of the flip alternatives observed had the lip below the modeled tailwater elevations. This either caused the flow to be unstable when it left the bucket or caused the flip to be suppressed. Both effects prevent the flow from impinging far downstream of the secondary basin, as desired. (Stantec Consulting Services, 2015)

It was also noted by the experts that “The performance of the Flip alternatives differs substantially from the performance of Super Baffle Alternative (RMP 6). Energy in the Flip alternatives is dissipated in a controlled manner until the hydraulic jumps in the stilling basins are swept out. At that point in time, energy dissipation in the stilling basin ceases, and all the energy is suddenly transferred downstream of the stilling basin. Such sudden transfer of energy results in uncontrolled, sudden and significant scour downstream of the stilling basin. For example in the case of Flip 4 (one of the flip alternatives), a sudden increase in scour of about five times greater than the scour just prior to sweepout occurs; increasing from only 10 feet to about 60 feet.” (Stantec Consulting Services, 2015)

It was also advised that such sudden loss of control can lead to unexpected and uncertain consequences, which are undesirable. This is particularly relevant when accounting for the uncertainty associated with the formation of the scour hole. It is reasonable to expect that the maximum depth of the prototype scour hole may be located closer to the stilling basin and may be deeper than observed in the physical model. It is conceivable that such sudden and dramatic increase in scour depth could potentially result in failure of the rock slope immediately downstream of the stilling basin, with possible headcutting towards the dam as the stilling basin potentially fails. Should such headcutting occur, it could endanger the dam. For this reason, additional scour countermeasures would need to be provided for reliability of the energy dissipator once the transition has occurred. This would likely be in the form of a cutoff wall or a preformed scour hole downstream of the flip.

5.2.6.3.6. Environmental Evaluation

In order to reduce the uncertainty in performance of this alternative, either a preformed scour hole or a concrete cutoff wall downstream of the existing stilling weir apron would be required for it to be constructed. Construction of RMP 5 would result in significantly greater impacts to the Resource category 1 habitat. The U.S. Fish and wildlife service mitigation policy, established in accordance with the Fish and Wildlife Coordination Act (16 U.S.C 661-667(e)) and the National Environmental Policy Act (42 U.S.C. 4321-4347) defined resource categories to assist in determining appropriate mitigation goals for federal water resource development projects. According to the policy, the designation criteria for habitat in Resource Category 1 is "habitat to be impacted is of high value for evaluation species and is unique and irreplaceable on a national basis or in the ecoregion section." The mitigation goal for habitat in Resource Category 1 is "no loss of existing habitat value." As there are other reasonable alternatives available which would meet objectives and avoid permanent impact to this high value habitat, this alternative is considered unacceptable in terms of applicable laws, regulations and public policies.

5.2.6.3.7. Evaluation for RMP 5 Evaluation

Significant concerns about the effectiveness and technical feasibility of RMP 5 were identified, especially when compared to other alternatives which achieve risk management objectives (tolerable risk guidelines). Secondarily, there would be permanent adverse effect to Resource Category 1 habitat, or other sensitive habitat associated with the construction of this more stilling basin that are avoided by other more reasonable alternatives. As other more reasonable alternatives exist to meeting the study objectives, further consideration of RMP 5 is eliminated.
5.2.6.4. Evaluation of RMP 6 - Stilling Basin with Super-cavitating Baffles

5.2.6.4.1. Effectiveness Evaluation

There is a high degree of confidence that RMP 6 reduces incremental risks associated with PFM 33 well below TRG and meets the study objective. A quantitative risk assessment (reported in Chapter 7) was completed for this plan and confirms this RMP is below TRG making it effective. See Figure 7-1 for details of the results.

The parapet wall was evaluated for further risk reduction as part of RMP 6 as an ALARP consideration. ALARP considerations provide a way to reduce risk below the tolerable risk limit until such actions are impracticable or not cost effective. The parapet wall as previously described was designed to complete the partially built 8-ft wall from the DSA project for $3M to $4M. The parapet wall option was developed to provide an incremental risk reduction up to one order of magnitude below PFM 35 for APF and AALL. This measure was evaluated based on the four components of ALARP shown in Table 5-6.

An analysis of the abutment monoliths (PFM 35) was done for both with and without a parapet wall. The analysis showed there is basically no risk reduction provided by the parapet wall. The analysis with the parapet wall was essentially the same as the abutment monolith stability analysis done for the ECRA. The main differences are that the probability of downstream scour was included in the analysis to simplify the event tree and for the with parapet analysis, the top of dam was EL 1,543-ft. For the with parapet wall analysis, the probability of downstream scour was based on the no debris blockage scenario elicited for the ECRA. This is because the parapet wall would be placed upstream of the handrail which the elicitation panel for the ECRA felt could collect debris. In order to make an apples to apples comparison of the results, the without parapet wall scenario was also analyzed including the probability of downstream scour within the analysis. For the without parapet wall analysis, the probability of downstream scour was based on the debris blockage scenario just like the ECRA assessment. Figure 5-3 shows the SRP for both the with and without parapet wall scenarios. After evaluating the risk reduction provided by the parapet wall as shown in Table 5-6, it was determined that it provided little to no risk reduction; therefore, it was eliminated from further consideration and determined impracticable.

While there is a sizeable reduction in likelihood of breach between pool EL 1,535-ft and EL 1,546-ft, there is essentially no reduction between EL 1,550-ft and the IDF. Due to the remote likelihood of these events, the total risk associated with this failure mode is driven by the upper portion of the loading curve. In other words, storm events resulting in EL 1,550-ft and above are driving the risk. See Figure 5-4 for an f-N chart comparing the with and without parapet wall scenarios. It is this lack of risk reduction at the IDF which is driving the lack of risk reduction for the parapet wall. The risk associated with the abutment monoliths is being driven by the IDF for both the with and without parapet wall scenarios. Since there is essentially no reduction in the risk at the IDF with a parapet wall, the significant reduction in likelihood of breach at EL 1,542-ft has minimal impact to the overall risk. It should be noted that the assumption that debris getting caught up in the handrail (debris blockage scenario) is capable of reducing overtopping flows for the first three feet of overtopping had minimal impact on the risk calculations. As stated previously, the risk associated with the abutment monoliths is being driven by the IDF. During the IDF, the dam is being overtopped by approximately 17 feet without a parapet wall and 10 feet with a parapet wall. After the pool rises approximately 7 feet above any obstruction to overtopping, whether it be a 3 foot debris dam or an 8 foot parapet wall, the maximum amount of scour behind the abutment monoliths is expected to be reached. Seven feet of overtopping for the with parapet wall scenario would occur at EL 1,550-ft. Seven feet of overtopping for the without parapet wall scenario would occur at pool EL 1,545-ft with the debris blockage assumption. This means the SRP for both scenarios for the IDF would be unchanged even if the debris blockage assumption was removed from the analysis.
The combination of lack of risk reduction provided by the parapet wall and the fact that very infrequent flood events must occur in order for benefits associated with the parapet wall to be realized makes it difficult to justify inclusion of the parapet wall. While there are other considerations for ALARP than just risk reduction, such as societal concerns with its exclusion and compliance with essential USACE guidelines, their benefits do not outweigh the lack of risk reduction. Therefore it was eliminated from further consideration and determined impracticable. The cost-to-save-a-statistical life (CSSL) was not calculated for this evaluation due to the lack of risk reduction and the determination of being impracticable. For more discussion on the parapet wall analysis, see the TSP documentation in Appendix A.

![Graph showing system response for with and without parapet wall](image)

**Figure 5-3: Abutment Monolith System Response for With and Without Parapet Wall**

<table>
<thead>
<tr>
<th>Potential Failure Mode (PFM)</th>
<th>APF (with parapet)</th>
<th>APF (without wall)</th>
<th>AALL (with wall)</th>
<th>AALL (without wall)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PFM 33 (Spillway Monolith Instability)</td>
<td>6.74E-8</td>
<td>2.92E-8</td>
<td>4.09E-5</td>
<td>1.71E-5</td>
</tr>
<tr>
<td>PFM 34 (Non-overflow Monolith Instability)</td>
<td>6.81E-8</td>
<td>6.50E-8</td>
<td>4.15E-5</td>
<td>3.82E-5</td>
</tr>
<tr>
<td>PFM 35 (Abutment Monolith Instability)</td>
<td>6.49E-7</td>
<td>7.58E-7</td>
<td>3.99E-4</td>
<td>4.51E-4</td>
</tr>
<tr>
<td>Total</td>
<td>7.84E-7</td>
<td>8.52E-7</td>
<td>4.81E-4</td>
<td>5.06E-4</td>
</tr>
</tbody>
</table>

Table 5-6: Comparison of RMP 6 with and without 8-ft Parapet Wall
Figure 5-4: Risk Plot for the Parapet Wall Measure
Table 5-7: Eight Foot Parapet Wall – ALARP Evaluation

<table>
<thead>
<tr>
<th>Level of Risk in relation to Tolerable Risk Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>ECRA assumes debris dam buildup preventing substantial overtopping (PFM 34 &amp; 35) above current top of dam, EL 1,535-ft. Debris dam assumptions creates some uncertainty for the loading increment just above top of dam. However, <strong>the total risk is not affected since it is driven by the upper end of the loading curve, which is not affected by the debris.</strong></td>
</tr>
<tr>
<td>May help mitigate for the uncertainty in hydrology analysis which is estimated to pass ~ 70% of PMF prior to overtopping the dam (EL 1535-ft) without the parapet wall and ~ 80% of the PMF (EL 1542-ft) with the parapet wall.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Cost Effectiveness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relatively low cost ($3-4M) item compared to other measures to reduce overtopping risk. Because this component was part of the original DSA, there is relatively little cost uncertainty with implementation of remaining 8-ft wall components.</td>
</tr>
<tr>
<td>A portion of the wall is already completed (i.e. gate closure and a few panels), remaining investment is relatively small.</td>
</tr>
<tr>
<td>Provides more flexibility (time to make decision) in operation of the auxiliary spillway penstock gates. May be able to delay opening and not damage the penstock gates.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Good Practice</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parapet wall more certain to function to prevent overtopping than debris buildup on handrail and functioning similarly to parapet wall as assumed in the ECRA.</td>
</tr>
<tr>
<td>Provides more time to actively perform operations such as gate opening and waterproofing when preparing for a flood. Additional time to access the top of dam with the additional 8-ft.</td>
</tr>
<tr>
<td>Provides more flexibility (time to make decision) in operation of the auxiliary spillway penstock gates. May be able to delay opening and not damage the penstock gates.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Societal Concerns</th>
</tr>
</thead>
<tbody>
<tr>
<td>Provided additional opportunity to reduce non-breach risk through future water control manual changes to modify surcharge. However, holding additional water behind the dam could potentially increase the incremental consequences.</td>
</tr>
<tr>
<td>Portions of the population downstream are at or below the poverty level. This population may be considered more vulnerable as they have less ability to recover from flood events.</td>
</tr>
</tbody>
</table>

**5.2.6.4.2. Efficiency Evaluation**

A construction cost estimate was developed for the conceptual design and resulted in approximately $300-$500M (for final estimate see Section 6.5).

**5.2.6.4.3. Acceptability Evaluation**

Construction of RMP 6 would likely result in significant temporary impacts to Resource category 1 habitat. The U.S. Fish and wildlife service mitigation policy, established in accordance with the Fish and Wildlife Coordination Act (16 U.S.C 661-667(e)) and the National Environmental Policy Act (42 U.S.C. 4321-4347) defined resource categories to assist in determining appropriate mitigation goals for federal water resource development projects. According to the policy, the designation criteria for habitat in Resource
Category 1 is "habitat to be impacted is of high value for evaluation species and is unique and irreplaceable on a national basis or in the ecoregion section." The mitigation goal for habitat in Resource Category 1 is "no loss of existing habitat value."

As all other alternatives considered would cause more severe environmental impacts, this alternative was considered most acceptable to in terms of applicable laws, regulations and public policies.

5.2.6.4.4. Completeness Evaluation
This plan is considered complete as it provides and accounts for all necessary investments to ensure the realization of the DSMS risk management objectives.

5.2.6.4.5. Engineering (Technical) Feasibility Evaluation
This alternative allows dissipation of energy in a controlled manner throughout the entire range of flows tested. The scour downstream of the stilling basin is expected to gradually increase as discharge through the stilling basin increases. The amount of uncertainty related to consequences of scour is low. RMP 6 is a more traditional stilling basin than the transitional flip and is very similar to the downstream conventional stilling basin in energy dissipation approach and performance. It is worth noting, that the performance of the second stage energy dissipator is not conventional. The basin was originally designed to be less than the jump length and to have super critical flow exiting the basin at higher discharges. This issue was not fully understood during the final array screening; but was further evaluated in the development of the selected plan and believed not to be a major technical concern.

RMP 6 has fewer constructability concerns compared to other alternatives except for RMP 1 and RMP 7. The major quantities were developed for RMP 6 and can be found in Table 5-8. These quantities were used to develop the cost estimate as discussed above.

Table 5-8: Major Quantities for RMP 6

<table>
<thead>
<tr>
<th>Material</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>100,000 CY</td>
</tr>
<tr>
<td>Reinforcing Steel</td>
<td>Between 15 and 20 Million Pounds</td>
</tr>
<tr>
<td>Rock Excavation</td>
<td>75,000 CY</td>
</tr>
<tr>
<td>Common Excavation</td>
<td>58,000 CY</td>
</tr>
<tr>
<td>Random Fill</td>
<td>36,000 CY</td>
</tr>
<tr>
<td>Cofferdam</td>
<td></td>
</tr>
<tr>
<td>Steel Sheet Piling</td>
<td>114,000 SF</td>
</tr>
<tr>
<td>Granular Fill</td>
<td>12,200 CY</td>
</tr>
<tr>
<td>Stone Protection</td>
<td>2,000 CY</td>
</tr>
<tr>
<td>Rockfill</td>
<td>20,000 CY</td>
</tr>
<tr>
<td>Anchors (Strand and Bar)</td>
<td>Approximately 1,900 EACH</td>
</tr>
</tbody>
</table>

5.2.6.4.6. Environmental Evaluation
This plan has been potentially found to be the Least Environmentally Damaging Practicable Alternative (LEDPA). All analysis done for this plan can be found in the SFEIS located in Appendix K.

5.2.6.4.7. Conclusion for RMP 6 Evaluation
RMP 6 without the parapet wall is the most effective and efficient plan within the final array that meets the goals and objectives of the study. RMP 6 is also the LEDPA.
5.2.6.5. RMP 7 - Concrete Overlay of Exposed Rock in Stilling Basin

5.2.6.5.1. Effectiveness Evaluation

Although this plan is intended to target the three main vulnerabilities associated with PFM 33, the plan is not designed as a system of components working together to improve the performance of the stilling basin. Because of this, there are likely additional vulnerabilities that could be exposed or design details which may not be feasibly overcome. For instance, the hydraulic performance of the basin with the current baffle and weir configuration creates a plunging flow at the upstream face of the weir. Although the concrete pavement may protect the rock, the joint between the pavement and weir would be subjected to extremely high pressure fluctuations which could create jacking forces beneath the pavement which could lead to displacement of the slab. The plan also does not address stability issues and overtopping of the training walls which could also initiate a path to dam breach.

A risk assessment was not completed for this plan. Due to the potential vulnerabilities associated with this plan, the risks associated with it would have such a significant uncertainty to make it difficult to conclude the risks were below the tolerable limit. Features could be added to this plan to overcome these uncertainties; however, the addition of these features would look essentially the same as RMP 6. Therefore, the conclusion is this plan would not meet the tolerable risk limit for life safety.

5.2.6.5.2. Efficiency Evaluation

Due to this plan not meeting the tolerable risk limit for life safety a cost estimate was not completed. The cost of this plan is expected to be least expensive when compared to the other plans in the initial array besides the FWAC (No Action) plan. However, this plan includes many of the same features as RMP 6 and requires similar construction which would result in costs of a similar order of magnitude. Yet, given this plan would not reduce risks below the tolerable limit it would be much less efficient than other plans.

5.2.6.5.3. Acceptability Evaluation

This plan would provide for and improvement from the 2016 ECRA and FWAC. However, it is not acceptable because risks would not be within the agency’s tolerable risk guidelines.

5.2.6.5.4. Completeness Evaluation

Does not address all the components associated with the failure mode. It addresses the scour in the unlined portion of the stilling basin, but does not address scour due to instability of the other existing features. The plan is incomplete as it does not account for all necessary investments or other actions to ensure the realization of the DSMS risk management objectives (TRG).

5.2.6.5.5. Engineering (Technical) Feasibility Evaluation

A quantitative risk assessment was not completed on this plan because it was determined at the beginning of the analysis that it did not meet tolerable risk guidelines. This plan was evaluated qualitatively and was determined that it does not improve the hydraulics of the stilling basin. As this plan did not meet the goals and objectives of the DSMS, quantities were not developed.

5.2.6.5.6. Environmental Evaluation

Minimal environmental impacts would occur with this plan as all permanent features are located within the stilling basin in the existing dam footprint. However, without additional investments to lower life-safety risks from Bluestone Dam, the dam would potentially breach leaving the safety of the surrounding human and natural environment severely impacted. There would also be subsequent effects to the local and regional economies. Some of the environmental impacts that could occur as a result of Bluestone Dam breaching include significant soil displacement, sedimentation, scouring, habitat destruction including wetlands, significant reduction in species abundance and diversity, flora and fauna mortality,
impacts to both up and downstream recreational facilities and fishing quality, water quality impacts, significant downstream impacts to cultural resources, and significant risk to public safety downstream of the dam.

5.2.6.5.7. Conclusion for RMP 7 Evaluation

RMP 7 was eliminated as it did not meet the goals and objectives of the DSMS.

5.2.7. Selection of Tentatively Selected Plan

Following acceptance of the 2016 ECRA and FWAC, formulation efforts were focused on addressing PFM 33 in consideration of ALARP to further reduce life-safety risks associated with PFM 34 and 45 with completion of a previously approved and partially constructed eight foot parapet wall. RMP 6 without the 8-foot parapet wall was found to be the most effective and efficient plan, meeting the goals and objectives of the DSMS without causing significant permanent environmental impacts. Figure 5-5 shows a rendered aerial view of RMP 6 after construction is complete.

Figure 5-5: Rendered Aerial View of RMP 6 Completion
Figure 5-6: RMP 4 - Downstream Conventional Stilling Basin Concept (Evaluation & Comparison Stage)

(Note: The design details shown in the figure may have changed as the formulation process progressed.)
Figure 5-7: RMP 4 - Downstream Conventional Stilling Basin Plan View

(Note: The design details shown in the figure may have changed as the formulation process progressed.)
Figure 5-8: RMP 4 - Downstream Conventional Stilling Basin Cross Sectional View (Evaluation & Comparison Stage)

(Note: The design details shown in the figure may have changed as the formulation process progressed.)
Figure 5-9: RMP 5 - Transitional Flip Stilling Basin Conceptual Design (Evaluation & Comparison Stage)

(Note: The design details shown in the figure may have changed as the formulation process progressed.)
Figure 5-10: RMP 5 - Transitional Flip Stilling Basin Plan View (Evaluation & Comparison Stage)

(Note: The design details shown in the figure may have changed as the formulation process progressed.)
Figure 5-11: RMP 5 - Transitional Flip Basin Cross Sectional View (Evaluation & Comparison Stage)
(Note: The design details shown in the figure may have changed as the formulation process progressed.)
Figure 5-12: RMP 6 - Conceptual Design (Evaluation & Comparison Stage)

(Note: The design details shown in the figure may have changed as the formulation process progressed.)
Figure 5-13: RMP 6 - Super-cavitating Baffle Stilling Basin Plan View (Right Side Construction)

(Note: The design details shown in the figure may have changed as the formulation process progressed.)
4. New reinforced concrete slab (10’ thick) with anchors.
5. New reinforced concrete slab protection (10’ thick) to be anchored.
6. Existing concrete drainage base to be removed and replaced with new more robustly reinforced concrete.
7. New reinforced concrete slab 7’ thick with protection to be anchored.
8. New reinforced concrete slab 10’ thick with protection to be anchored.
10. For details of drainage system, structural, and anchors.
Figure 5-15: RMP 6 - Stilling Basin with Super-cavitating Baffles Cross Sectional View
Figure 5-16: RMP 7 - Concrete Overlay Basin Plan View

(Note: The design details shown in the figure may have changed as the formulation process progressed.)
Figure 5-17: RMP 7 - Concrete Overlay Cross Sectional View

(Note: The design details shown in the figure may have changed as the formulation process progressed.)
## Table 5-9: Evaluation and Comparison of Risk Management Plans

<table>
<thead>
<tr>
<th>RMP 1</th>
<th>RMP 4</th>
<th>RMP 5</th>
<th>RMP 6</th>
<th>RMP 7</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Effectiveness (Incremental Risk Reduction)</strong></td>
<td>Not effective. There will be minimal risk reduction beyond DSA construction. Risks would remain above tolerable risk guidelines.</td>
<td>Effective. Expected to reduce risk below tolerable risk guidelines</td>
<td>Effectiveness uncertain. Significant scour would occur downstream of flip component introducing uncertainty in performance</td>
<td>Effective. Expected to reduce risk below tolerable risk guidelines</td>
</tr>
<tr>
<td><strong>Efficiency (Cost)</strong></td>
<td>$15M to install 66 +/- dam anchors to meet risk condition estimated in 2016 ECRA.</td>
<td>This plan is expected to be between $400M and $700M.</td>
<td>Plan was screened due to the uncertainty of effectiveness, therefore a cost estimate was not developed.</td>
<td>This plan is expected to be between $300M and $500M.</td>
</tr>
<tr>
<td><strong>Acceptability</strong></td>
<td>Not acceptable because risks would not be within the agency's tolerable risk guidelines.</td>
<td>As there are other reasonable alternatives available which would meet objectives and avoid permanent impact to this high value habitat, this alternative is considered unacceptable in terms of applicable laws, regulations and public policies</td>
<td>As there are other reasonable alternatives available which would meet objectives and avoid permanent impact to this high value habitat, this alternative is considered unacceptable in terms of applicable laws, regulations and public policies</td>
<td>Acceptable; Least Environmentally Damaging Practicable Alternative</td>
</tr>
<tr>
<td><strong>Completeness</strong></td>
<td>Not Complete</td>
<td>Complete</td>
<td>Complete</td>
<td>Complete</td>
</tr>
<tr>
<td><strong>Engineering (Technically Feasible)</strong></td>
<td>It is constructible. A work platform will have to be installed across the primary spillway</td>
<td>It is constructible. However, there are challenges with constructing the cofferdam due to the height requirement caused by the deep excavation.</td>
<td>It is constructible, but additional measures may be required to address scour concerns such as a large cutoff wall or a pre-formed scour hole. There are uncertainties and constructability challenges if these measures are required.</td>
<td>It is constructible. It has fewer constructability concerns compared to other alternatives with the exception of FWAC (No Action) plan and the concrete overlay.</td>
</tr>
<tr>
<td><strong>Environmental Impacts</strong></td>
<td>Minimal from installation of dam anchors However, if breach were to occur, impacts would be significant and permanent</td>
<td>Significant permanent impacts due to construction in a Resource category 1 habitat.</td>
<td>Significant permanent impacts due to construction in a Resource category 1 habitat only if measures (cutoff wall or pre-formed scour hole) are required to address scour.</td>
<td>Minimal environmental impact. Permanent features are within the existing dam footprint. Temporary impact of Resource Category 1 habitat</td>
</tr>
<tr>
<td><strong>Conclusion of Screening process</strong></td>
<td>Eliminated from further consideration. Does not meet objectives (tolerable risk guidelines).Retained for comparison purposes.</td>
<td>Eliminated. Inefficient compared to other plans and effects to Resource Category 1 habitat would be significant and permanent</td>
<td>Eliminated. Technically challenging with uncertain effectiveness and effect to Resource Category 1 habitat could be significant and permanent</td>
<td>Selected. Only RMP that meets goal and objectives of DSMS effectively, efficiently and without causing significant permanent environmental impacts</td>
</tr>
</tbody>
</table>

1 June 2017
### Table 5-10: Quantitative Evaluation and Comparison of Final Array of Plans

<table>
<thead>
<tr>
<th>Costs</th>
<th>No Action</th>
<th>Alternatives</th>
<th>Existing FWAC</th>
<th>RMP 4: Downstream Conventional</th>
<th>RMP 5: Transitional Flip</th>
<th>RMP 6: Super Baffle “TSP”</th>
<th>RMP 7: Concrete Overlay</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Construction</td>
<td></td>
<td>$773,000,000</td>
<td>$188,000,000</td>
<td>$160,000,000</td>
<td>$157,000,000</td>
<td>$84,955,000</td>
</tr>
<tr>
<td></td>
<td>Non-Construction (PEC, S&amp;A, EDC)</td>
<td></td>
<td>$188,000,000</td>
<td>$146,000,000</td>
<td>$160,000,000</td>
<td>$157,000,000</td>
<td>$84,955,000</td>
</tr>
<tr>
<td></td>
<td>Contingency (%)</td>
<td></td>
<td>35%</td>
<td>35%</td>
<td>30%</td>
<td>35%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Project Cost (Current FY)</td>
<td></td>
<td>$1,660,000</td>
<td>$1,830,000</td>
<td>$1,830,000</td>
<td>$1,830,000</td>
<td>$1,830,000</td>
</tr>
<tr>
<td>Dam Performance</td>
<td>O&amp;M Costs (annual)</td>
<td></td>
<td>$600,000</td>
<td>$1,740,000</td>
<td>$1,830,000</td>
<td>$1,830,000</td>
<td>$1,830,000</td>
</tr>
<tr>
<td></td>
<td>Annualized Probability of Failure (APF)</td>
<td></td>
<td>1.08E-05</td>
<td>7.40E-06</td>
<td>8.52E-07</td>
<td>8.52E-07</td>
<td>8.52E-07</td>
</tr>
<tr>
<td>Incremental Social/Economic Consequences of a Breach</td>
<td>Societal Annualized Life Loss (ALL)</td>
<td></td>
<td>2.20E-02</td>
<td>6.14E-03</td>
<td>5.06E-04</td>
<td>5.06E-04</td>
<td>5.06E-04</td>
</tr>
<tr>
<td>Flood Damages (Range of All Loading Conditions)</td>
<td>Incremental Social/Economic Consequences of a Breach</td>
<td></td>
<td>$33,000</td>
<td>$21,000</td>
<td>$2,221</td>
<td>$2,221</td>
<td>$2,221</td>
</tr>
<tr>
<td></td>
<td>SRMFR</td>
<td></td>
<td>$6,758,357,753</td>
<td>$3,697,514,526</td>
<td>$3,492,289,792</td>
<td>$2,868,752,497</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Cost to See a Statistical Life (Annual)</td>
<td></td>
<td>$38,035,367</td>
<td>$20,850,576</td>
<td>$19,634,340</td>
<td>$11,889,135</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Benefit Cost Ratio (BCR)</td>
<td></td>
<td>0.000</td>
<td>0.001</td>
<td>0.001</td>
<td>0.001</td>
<td></td>
</tr>
<tr>
<td>Other Factors</td>
<td>ALARP considerations included?</td>
<td>YES/NO</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Meets Essential USACE Guidelines?</td>
<td>YES/NO</td>
<td>Yes</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Does the alternative increase non-breach risk?</td>
<td>YES/NO</td>
<td>Yes</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Does the alternative cause environmental damages?</td>
<td>YES/NO</td>
<td>Very high as permanent features beyond existing footprint. Permanent impact to Resource Category 1 habitat.</td>
<td>Low, Permanent features within the existing project footprint. Temporary impacts to Resource Category 1 habitat.</td>
<td>Low, Permanent features within the existing project footprint. Temporary impacts to Resource Category 1 habitat.</td>
<td>Low, Permanent features within the existing project footprint. Temporary impacts to Resource Category 1 habitat.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Completeness</td>
<td>YES/NO</td>
<td>Yes</td>
<td>Yes</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Acceptable</td>
<td>YES/NO</td>
<td>No. Permanent, significant environmental impacts.</td>
<td>Yes</td>
<td>Yes</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Robustness</td>
<td>High, Medium, Low</td>
<td>High</td>
<td>High</td>
<td>High</td>
<td>Low</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Redundancy</td>
<td>High, Medium, Low</td>
<td>High</td>
<td>High</td>
<td>High</td>
<td>Low</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Resilience</td>
<td>High, Medium, Low</td>
<td>High</td>
<td>Low</td>
<td>Medium</td>
<td>Low</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Implementation Duration (PED &amp; Construction)</td>
<td>Years</td>
<td>15-17 years</td>
<td>15-15 years</td>
<td>15-15 years</td>
<td>9-11 years</td>
<td></td>
</tr>
</tbody>
</table>
5.2.8. Principles and Guidelines Accounts

The final array of alternatives was evaluated on impacts to society, the economy and the environment. The four accounts as established in the Economic and Environmental Principles for Water and Related Land Resources Implementation Studies were used to facilitate evaluation and display of effects of alternative plans.

These accounts include:

- Other Social Effects (OSE) - societal and individual health and human safety,
- National Economic Development – net value of the national output of goods and services,
- Regional Economic Development – regional economic activity,
- Environmental Quality - non-monetary effects on significant natural and cultural resources.

Each of these accounts have a material bearing on the decision making process; however, the primary driver for plan selection and justification of federal action is part of the Other Social Effects account and involves life safety.

5.2.8.1. Other Social Effects

The primary goal and objective of the Bluestone DSMR is to identify alternatives that reduce expected life loss from a dam failure to a level that is within Tolerable Risk Guidelines (TRG). There are also other social benefits provided by reducing the probability of failure of Bluestone, including psychological well-being; and continued educational, cultural and recreational opportunities. The following considerations and executive orders were considered in the evaluation of RMPs and details can be found in Appendix H. Given all the plans reduce incremental risk of dam failure, no segment of the population would be disproportionately affected by any of the RMPs in the final array.

- Life Safety
- E.O. 11593, Protection and Enhancement of the Cultural Environment
- E.O. 12898, Environmental Justice
- E.O. 13045, Protection of Children
- Community and Social Cohesion

5.2.8.2. National Economic Development

NED benefits are defined as increases in the economic value of the goods and services that result directly from a project. These are benefits that occur as a direct result of the project and are national in perspective. Benefit categories considered by the analysis include recreation and flood risk management (flood control). These categories represent important national considerations. NED is not being used for evaluation and comparison criteria for the RMPs because the primary criteria for the Bluestone Dam Safety Modification Study is based on life safety (reduction in incremental life loss). National economic development benefits were calculated for the final array of plans and the benefit-to-cost-ratio (BCR) was approximately 0.001 for each plan in the final array (See Table 5-10). Economic benefits were not used to justify the selection of a plan. The TSP was selected and justified based on life safety benefits.

5.2.8.2.1. Flood Risk Management

All plans in the final array are similar in that they reduce the annualized probable incremental economic consequences associated with the project. Breach and non-breach damages are the same for each plan, but these damages and the loss of flood risk management benefits associated with a breach are annualized.
in DAMRAE based on the system response probabilities. Improvements in the system response result in a reduction of the annualized economic consequences associated with project risk.

### 5.2.8.2.2. Recreation

The effect of the proposed alternatives on outdoor recreation has been evaluated as required under the Federal Water Project Recreation Act of 1965, As Amended. All plans in the final array adversely affect existing recreational opportunities. These impacts are evaluated in the SFEIS.

In addition, the probability of experiencing a breach and resulting substantial impacts on recreational opportunities will be greatly reduced with the implementation of any of the alternatives. Future lost recreation benefits due to a breach are included in the overall economic consequences of a breach (which are then annualized in DAMRAE according to the system response). This project complies with the goals of the Recreation Act.

### 5.2.8.2.3. Hydropower

Risk management plans developed for this study to address the PFM’s presented do not preclude hydropower development at the dam.

### 5.2.8.3. Environmental Quality

The EQ account is used to present non-monetary effects on ecological, cultural, and aesthetic resources including the positive and adverse effects of ecosystem restoration plans.

E.O. 11514 directs Federal agencies to “initiate measures needed to direct their policies, plans, and programs so as to meet national environmental goals.” While the primary goal of the remediation of Bluestone Dam is to ensure that expected annual life loss is below tolerable risk guidelines, objectives of the project also include lowering the probability of experiencing a breach.

The probability of experiencing a breach and incurring substantial impacts on ecological, cultural and aesthetic resources will be greatly reduced with the implementation of any of the alternatives. Adverse effects associated with implementing any of the alternatives are expected to be minimal to moderate. Many effects, such as recreation and noise levels would be temporary during construction activities. Bluestone Dam modification as a whole is not expected to significantly affect protected species.

All alternatives may affect, but are not likely to adversely affect, the threatened and endangered species and their critical habitat in the project area. For all alternatives, species would not be directly affected by construction of any of the alternatives in the final array; however, there is potential for disturbance to threaten and endangered (T&E) species during construction activities. Additional detail can be found in Appendix K.

### 5.2.8.4. Regional Economic Development

#### 5.2.8.4.1. Flood Insurance Impacts

None of the plans in the final array influence the current eligibility status for downstream residents in regards to the National Flood Insurance Program (NFIP) except for RMP 2.

#### 5.2.8.4.2. Floodplain Management

E.O. 11988 directs Federal agencies to avoid siting projects in floodplains and to avoid inducing further development of flood-prone areas. The project is not a new flood risk remediation project but rather the maintenance of a previously authorized and constructed multi-purpose project. Through expanded state, municipal and public education about the risk associated with Bluestone Dam, it is believed that due diligence is provided to communicate the inherent risks of development in flood-prone areas.
5.2.8.4.3. Employment

The final array of plans, with exception of the FWAC (No Action) plan, RMP 2, and RMP 7 would provide an increased level of protection to commercial, industrial, governmental and agricultural jobs that may be permanently lost as a result of a dam failure. In addition, the construction of any recommended features would have a beneficial effect on employment and demand for local goods and services during the construction period.
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6. SELECTED PLAN (RMP 6)

After comparison of the effectiveness and efficiency of the final array of alternatives in meeting the project objectives, the basin with the super-cavitating baffles has been identified as the selected plan (RMP 6). RMP 6 is a cost-effective solution consisting of structural measures working in unison to reduce risk and achieve the primary objective of reducing the risk of life loss. RMP 6 will effectively reduce the risk of life loss to below tolerable risk guidelines for Bluestone Dam by addressing the risk associated with PFM 33, Spillway Monolith Instability.

6.1. Description and Overview of RMP 6

As previously noted, the purpose of RMP 6 is to reduce the risk associated with PFM 33, Spillway Monolith Instability. In addition to providing the additional anchors in the dam, the spillway design accomplishes incremental risk reduction by modifying the first stage stilling basin to prevent rock scour, stabilizing the stilling weir, reinforcing the second stage basin to minimize scour, and addressing scour concerns with overtopping of the training walls. Specific details of the design are described in the paragraphs below.

Based on risk informed decision making practices, it was determined the construction of the recommended plan will be implemented in two phases, Phase 5 and Phase 6 (if necessary). Phase 5 consists of the features described in RMP 6 including the construction of the under drains, but not including the construction of the toe gallery. Based on the best available information and conservative assumptions, the anticipated performance of RMP 6 with the underdrain system only (i.e., no toe gallery) is estimated to achieve tolerable levels of risk. However, there is substantial uncertainty associated with this conclusion. The information needed to confirm these findings will be obtained only when a substantial portion of the new stilling basin is put in service and data regarding uplift pressures during pool loading scenarios is acquired. If adequate uplift relief is not achieved, Phase 6, construction of the toe gallery would be implemented. The Dam Senior Oversight Group (DSOG) supported the recommendation of this phased implementation approach. Additional details regarding design and implementation of the recommended plan can be found in Chapter 6.

6.1.1. Hydrologic Overview

The design for the modifications outlined in the 1998 DSA Report were based on an inflow (IDF) equivalent to the PMF estimate developed in 1982. This resulted in a peak reservoir EL 1,542-ft, with a parapet wall in place to prevent overtopping. The current estimate of the PMF produces a headwater of EL 1,553-ft considering existing conditions of the dam. Even though the updated PMF would result in a much higher reservoir elevation, the risks for the overtopping PFMs (PFMs 34 and 35) did not warrant action, even without the parapet wall in place. The DSA process, around which the original modifications were formulated, did not involve risk informed decision making. When taking into consideration the risks associated with these more extreme floods, the conclusions were the risks did not justify a modification to safely pass the full IDF.

Given the anchors within the dam were designed to a reservoir pool of EL 1,542-ft, consideration was given to maintaining the DSA Report design reservoir elevation. This results in a peak inflow of approximately 80% of the PMF; which has a best estimate annual chance exceedance of approximately 1 in 560,000. The resulting difference in discharge through the primary spillway between reservoir EL 1,535-ft and EL 1,542-ft is only about 30,000 cfs (less than 1% difference); due to flow restrictions provided by the nappe interception of the crane rail bridge beam and spillway gates. The biggest differences between the two pool elevations in the dam and stilling basin design loading will be the hydrostatic head and uplift due to the higher head. The risks associated with this design are assessed through the full range of loading and the results are presented in Chapter 7. However, there may be opportunities to optimize this design during PED while still achieving risk reduction objectives.
Considerations were also given to how the dam might be modified to pass the full IDF. The most likely modification to safely pass the full IDF would include not only a modified stilling basin, but also removal of restrictions to flow at the spillway crest, a parapet wall, and potentially a cut-off feature at the toe of the second stage basin to prevent undermining. This would result in a change to loading on the stilling basin features. Several features in the selected plan would require considerable demolition and reconstruction to modify the features to address the full IDF. Therefore, they are designed to the maximum potential discharge for the current estimate of the IDF. For a list of all pertinent RMP 6 features and how their design may be required to be modified to meet the full IDF loading, see Table 6-1.

6.1.2. Hydraulic Design Overview

The stilling basin design has undergone two physical model studies. The first assessment of RMP 6 was completed in 2014 with an evaluation of a 1:36 sectional model for proof of concept design. The second round of physical model study was performed in 2016, referred to as Phase A modeling, to develop RMP 6. This effort focused on evaluating the design through a range of flow conditions, developing estimates for loading based on sectional model measurements, and evaluating the wall raise requirements and flow interactions downstream of the stilling basin. This modeling effort was primarily qualitative and observational. A third round of physical modeling, referred to as Phase B modeling, will be completed as part of subsequent design efforts and will focus on improving hydraulic loading estimates and optimizing the final RMP 6 configuration.

The Phase A modeling results established several major recommendations as follows:

1. The stilling basin training walls do not need raised in order to have adequate performance of the first stage hydraulic jump. Protection to mitigate scour behind the training walls will likely be required but is expected to cost less than the wall raises.
2. The second stage stilling basin training walls do not require a raise.
3. The modeling of RMP 6 demonstrated adequate hydraulic performance up through the design condition, with no major changes in performance up to the current estimate of the IDF.
4. Undermining of the second stage stilling basin is not expected with a discharge from a reservoir EL 1,542-ft (to be confirmed in Phase B modeling).

The hydraulic design of RMP 6 is a modification of the existing two-stage hydraulic jump energy dissipator to accommodate flows associated with a headwater at EL 1,542-ft and a range of expected tailwaters. The first stage hydraulic jump energy dissipator utilizes a single row of super-cavitating baffle blocks. Baffle block size and configuration was optimized through iterative testing in Phase A modeling and additional cost-saving refinements will be explored during Phase B modeling.

The design of the super-cavitating baffle blocks was developed based on testing and design associated with the Folsom Dam auxiliary spillway (Frizell, 2009), and were designed to prevent major damage associated with high velocity flow resulting from cavitation erosion. There is also an intermittent ramp between the baffle blocks that is intended to direct vortices and swarms of cavitation bubbles to the surface to minimize cavitation damage on the stilling basin floor. The combination of the baffles and ramp allows the basin to function with a lower tailwater than typically achieved in baffled hydraulic jump stilling basins. In this aspect, the design is a highly forced hydraulic jump. Testing performed with the 1:65 scale general model demonstrates that the existing stilling weir provides the requisite tailwater to prevent sweep out, excessive baffle impact, or surging up to and beyond the current IDF estimate.

The second stage basin performs as a hydraulic jump stilling basin for range of flows up to approximately 200,000 cfs throughout the range of expected tailwater conditions. Above this range, super-critical flow exits the basin for low tailwater conditions. At the design headwater of EL 1,542-ft, super critical flows
exits the second stage at the full range of expected tailwaters. The original design acknowledged the super critical exit and optimized the stilling basin to efficiently minimize scour downstream of the endsill. Testing and interpretation of a movable bed model has demonstrated that the existing design is effective at preventing undermining of the existing endsill (Dingrando, PE, PG & Keeling, PE, CFM, 2016) up to the discharge associated with pool EL 1,542-ft.

Considering suggestions from independent hydraulic design experts (LRD Dam Safety Production Center, 2016), an evaluation of reducing/eliminating the first and second stage wall raises was performed using the 1:65 scale model during Phase A modeling. Observational evaluation of the 1:65 scale model demonstrated that satisfactory performance was achieved without a raise to either the first or second stage stilling basin walls; provided that overtopping flows are safely conveyed downstream of the structure.

6.2. Details of RMP 6 Features
Specific details regarding the design of RMP 6 are described in the following paragraphs. The main first stage features include the existing apron, super baffles, apron extension, divider wall with gallery, underdrain system, and training walls with overtopping scour protection. Other features of RMP 6 include the stilling weir, existing second stage stilling basin apron, toe drainage gallery, waterproofing, dam anchors, and non-structural measures. For general layout and basic description of pertinent features of RMP 6, see Figure 6-1, Figure 6-3, and Table 6-1. For reference purposes Figure 6-2 is included to show the existing dam view and profile. Further design details on primary features can be found in the following locations:

- **Appendix B – Structural Engineering**
  - Basin Stability – Sections 2 and 3
  - Global Stability – Section 4
  - Training Wall – Section 5
  - Toe Gallery – Section 6

- **Appendix C – Civil Engineering**
  - Construction Features – Section 2
  - TSP Quantities – Section 3

- **Appendix D – Hydrology and Hydraulics**
  - Phase A Physical Modeling – Attachment 7
  - Hydraulic Load Computations – Attachment 9

- **Appendix E – Geotechnical Engineering**
  - Failure Planes – Section 3
  - Rock Strength Parameters – Section 4
  - Foundation Drainage – Section 5
  - Anchor Design – Section 6 and 9
  - Rock Excavation and Foundation Preparation – Section 8

- **Appendix G – Electrical & Mechanical**
Figure 6-1: Overview of RMP 6 and Section View
Figure 6-2: 3D ISO View and Typical Profile of the Existing Bluestone Dam
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Figure 6-3: Typical Sections for Toe Gallery, Divider Wall Gallery, and Existing Cofferdam Removal in Penstock Basin
Figure 6-4: Typical Sections for Modified Stilling Basin
<table>
<thead>
<tr>
<th>Feature</th>
<th>Description</th>
<th>Reservoir Design Elevation (ft)*</th>
<th>Modification to full IDF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Existing Apron</td>
<td>Remove baffles and end sill, 5-ft concrete resurfacing, pre-stressed anchors, foundation drains</td>
<td>1,542</td>
<td>Additional anchors may be required due to increased uplift, but increase in water in basin could offset increased uplift</td>
</tr>
<tr>
<td>Super Baffles</td>
<td>16.5-ft x 11-ft</td>
<td>100% IDF</td>
<td>N/A</td>
</tr>
<tr>
<td>Super Baffle Monoliths</td>
<td>10 to 16.5-ft thick, pre-stressed anchors, foundation drains</td>
<td>1,542</td>
<td>Additional anchors are required</td>
</tr>
<tr>
<td>Apron Extension</td>
<td>10-ft thick, pre-stressed anchors upstream, pre-stressed anchors downstream, foundation drains</td>
<td>1,542</td>
<td>Additional anchors may be required due to increased uplift, but increase in water in basin could offset increased uplift</td>
</tr>
<tr>
<td>Divider Wall with Gallery</td>
<td>37-ft x 14-ft wall, 8-ft x 6-ft gallery</td>
<td>100% IDF</td>
<td>N/A</td>
</tr>
<tr>
<td>Training Walls</td>
<td>Pre-stressed anchors in walls, 7-ft thick concrete scour pad with passive anchors behind walls, foundation drains</td>
<td>1,542</td>
<td>Walls may require additional anchors, no change to scour protection as controlling case is likely less than 1542 due to lower tailwater</td>
</tr>
<tr>
<td>Stilling Weir</td>
<td>Pre-stressed anchors</td>
<td>1,542</td>
<td>Likely additional anchors</td>
</tr>
<tr>
<td>Existing Second Stage Baffles and End Sill</td>
<td>Remove and replace existing baffles and end sill</td>
<td>100% IDF</td>
<td>N/A</td>
</tr>
<tr>
<td>Existing Second Stage Apron</td>
<td>Pre-stressed anchors</td>
<td>1,542</td>
<td>Provisions to prevent undermining, likely some form of cut-off wall. Additional anchors may be required.</td>
</tr>
<tr>
<td>Waterproofing Dam Openings</td>
<td>Watertight openings</td>
<td>100% IDF</td>
<td>Would not be required if no overtopping flows</td>
</tr>
<tr>
<td>Dam Anchors</td>
<td>Pre-stressed anchors</td>
<td>1,542</td>
<td>Additional anchors may be required</td>
</tr>
<tr>
<td>Toe Gallery (Phase 6)</td>
<td>9.75-ft x 6-ft mined gallery, foundation drains, three adits to existing gallery</td>
<td>100% IDF</td>
<td>N/A</td>
</tr>
</tbody>
</table>

* Design reservoir elevation subject to optimization during PED
6.2.1. Existing Apron
The baffles, end sill and top five feet of the concrete floor of the existing apron will be removed leaving the top of the existing apron at EL 1,363-ft. Upon removal and replacement of the upper floor, anchors will be installed in the anchor head recesses of the new apron concrete. These will be active anchors required to improve flotation and sliding stability and will have Class I corrosion protection in accordance with the Post-Tensioning Institute (PTI) (Institute, 2014). Bearing plates for the anchors will be used to transfer anchor load to the new apron concrete.

Once all anchors are installed, stressed and locked off, one row of drains for the new underdrain system will be drilled at an angle of 15 degrees upstream and connected to the new piping grid plumbed to the new divider wall gallery.

Prior to placing new concrete for the floor of the existing apron, a top and bottom mat of reinforcement designed to span between anchor locations will be installed. Likewise, a double row of waterstops will be installed at all new joints of the apron and between the spillway and apron, training walls and apron, and divider wall and apron. For the spillway/apron joints and the training wall/apron joints, the waterstops will either be dove tailed cut and grouted into the spillway and training walls or a bolt on type waterstop will be used. The concrete placement for the finish floor will maintain the profile of the existing apron minus the baffles and end sill and will have a finish and joint details favorable for the anticipated high velocity flows in this area. The design of the existing apron includes doweling monoliths together and actively anchoring the new 5-ft thick concrete placement to improve its stability, its ability to provide passive resistance to the spillway monoliths and to resist hydraulic jacking. During PED, the PDT will explore other recommendations and considerations made by the expert panel on uplift and other alternatives to minimize risks and improve the design. Design refinements that will be considered in PED include raising the entire basin floor to minimize excavation damage to the existing concrete; using hydro-demolition for final excavation; and recognizing the need to formulate a concrete mix design (in consultation with a concrete material expert) for the new overlay to achieve strength and thermal compatibility with the existing concrete.

6.2.2. Super Baffles
RMP 6 includes the construction of 40 full and two partial baffles (adjacent to the divider wall) located just downstream of the existing apron. Also a special monolith will be required to include both the partial baffles and the adjacent divider wall. With the floor elevation of the baffle monolith at EL 1,368-ft, the founding elevation for the baffle monolith steps from EL 1,351.5-ft at the upstream end to EL 1,358-ft at the downstream end making the foundation thickness step from 16.5-ft to 10-ft. This 8-ft step provides passive sliding resistance for the baffle monolith.

In addition to the baffle weight and passive resistance of the step, sliding resistance for the baffle monoliths is accomplished via the installation of three rows of high strength steel strand anchors. Similar to the anchors in the existing apron, these anchors are required to improve sliding stability, will be active and will have Class I corrosion protection in accordance with PTI. Bearing plates for the strand anchors will be used to transfer anchor load to the baffle monolith prior to placing concrete in the anchor head recess.

Once all anchors are installed, stressed and locked off, one row of drains for the new underdrain system will be drilled, connected to the new piping grid, and plumbed to the new divider wall gallery. The new drains will be drilled 15 degrees from vertical upstream into rock.
The baffle blocks and their foundation will be heavily reinforced to resist anticipated hydrodynamic and impact loads. Also, all edges of the upstream face of each baffle will require embedded stainless steel armor for protection and to provide sharp corners necessary for proper function of the baffles. Waterstops will also be required for all joints of the baffle monoliths. In addition to the double row of waterstops already discussed for the existing apron/baffle monolith joint, a double row will be required for all joints between adjacent concrete structures both new and existing. As with the existing apron, for any joint involving existing concrete, the waterstops will either be dove tailed cut and grouted or a bolt on type waterstop will be used.

6.2.3. Apron Extension
The apron extension consists of upstream and downstream monoliths to protect the 158 feet of rock floor of the stilling basin from the downstream toe of the baffle monolith to the heel of the existing stilling weir. The planned height for the monoliths is 10 feet with a founding elevation of El. 1,358 and a top surface of EL 1,368-ft. Both the upstream and downstream monoliths will require four rows of active bar anchors to improve flotation and sliding stability that will have Class I corrosion protection in accordance with PTI. Bearing plates for the anchors will be used to transfer load to the top lift of concrete.

Once all anchors are installed, stressed and locked off, three rows of drains for the new underdrain system will be drilled and connected to the piping grid plumbed to the divider wall gallery. The drains will be drilled 15 degrees from vertical into rock.

Similar to the existing apron, a top and bottom mat of reinforcement designed to span between anchor locations will be installed in the apron extension. A double row of waterstops will be installed at all new joints of the upstream and downstream monoliths as well as all joints between adjacent concrete structures both new and existing. For any joint involving existing concrete, the waterstops will either be dovetailed cut and grouted or a bolt on type waterstop.

6.2.4. Divider Wall with Gallery
RMP 6 includes the construction of a stilling basin divider wall on the centerline of spillway monolith 34. This divider wall roughly splits the stilling basin in half, allowing for unwatering of one side of the basin, while maintaining dam operations on the other side. This configuration will allow for longer and more frequent inspection of the basin floor than currently possible. This longer duration allows for drain cleaning and basin maintenance as needed to ensure drain efficiency and proper basin function. This wall will also serve as a cofferdam during construction on the left side of the basin eliminating the need for an additional cellular cofferdam.

The divider wall is designed as a T-wall. The top of wall elevation is EL 1,405-ft, the top of the base is at EL 1,368-ft (finish floor of stilling basin), and the foundation of the base is EL 1,358-ft making the base 10 feet tall. The stem width is 14 feet allowing for construction of a gallery in its center. This gallery will be formed in with the construction of the stem and will be 6-ft wide by 8-ft tall (flat roof with chamfered corners) with a double gutter to tie into the underdrainage system of each half of the basin. Also, the floor elevation of the new divider wall gallery will be EL 1,370-ft with a sloped tunneled access to the existing inspection gallery (EL 1,375-ft). The gallery provides an efficient means of collecting flow from the underdrainage system, monitoring uplift instrumentation, and an access point for cleaning the underdrainage system.
A double row of waterstops will be used for all joints of the divider wall and as previously discussed, for all joints with adjacent features. Additionally, a double row of waterstops will be required around all joints of the gallery opening. As previously mentioned, the design of the divider wall monolith adjacent to the baffle monoliths will need to incorporate the construction of the two adjacent partial baffles. The profile of the foundation of this monolith will match that of the adjacent baffle monoliths, including a key at the upstream end to enhance sliding stability.

During the Phase A modeling, experiments were conducted on the 1:65 scale general model with the RMP 6 configuration, including the divider wall in the primary stilling basin. The configuration was tested with the full range of operational discharges and appeared to achieve adequate performance. Additional experiments are planned during Phase B modeling which will also include instrumentation on the divider wall with pressure transducers to better estimate the design loads.

### 6.2.5. Training Walls

Due to the increased discharge through the spillway above the original design, existing spillway and stilling basin training walls require modification to improve stability and protect from rock scour.

#### 6.2.5.1. Spillway Training Walls

The requirement for extensions of the existing spillway right and left training walls was identified in the Phase A modeling. The extensions are up to 10 foot high reinforced concrete walls that transition into the right and left training walls. The purpose of the extensions are to train all anticipated spillway flows preventing overtopping of the walls until reaching scour protection features associated with the basin walls.

#### 6.2.5.2. Right Stilling Basin Wall

Spillway discharges in conjunction with the baffle monoliths create a hydraulic jump in the apron extension of the first stage with a peak that is much higher than the existing training walls resulting in significant overtopping of the walls. The initial plan was to raise these walls to account for the overtopping and reduce the risk of rock scour at the heel of the walls. However, through investigation of the overtopping via the Phase A modeling in conjunction with expert observation of the model, an alternative plan has been developed to address the overtopping issue. Specifically, for the right training wall, the expert recommendation is to allow overtopping of the wall and provide rock scour protection at the wall heel. Therefore, the current plan to address the right wall overtopping is to anchor the wall for stability and provide a 7-ft thick concrete scour protection from the heel of the right spillway training wall to the heel of the left penstock training wall. Anchors required (to be refined in PED) to improve sliding stability for the right training wall will be two rows of high strength steel anchors per monolith in monoliths 56 through 62 and one row of high strength steel anchors per monolith in monoliths 63 and 64. These anchors will be active with Class I corrosion protection.

The concrete for scour protection will essentially cover all rock between the walls from monolith 56 to monolith 63 of the spillway training wall. The scour protection will also require anchors. These anchors are required to improve flotation stability, will be passive with Class I corrosion protection spaced at 10-ft on center. Other means of overtopping protection will be investigated during PED to reduce cost and construction schedule, while maintaining the required level of performance to meet risk reduction targets.

#### 6.2.5.3. Left Stilling Basin Wall

Similar to the right wall, the recommendation for the existing left wall is to allow overtopping and provide rock scour protection at the heel. Anchors required (to be refined in PED) to improve sliding stability for
the left training wall will be one row of high strength steel anchors per monolith for monoliths 100 through 98 and 93 through 91 and two rows of high strength steel anchors per monolith for monoliths 97 through 94. As with the right wall, the anchors will be active anchors with Class I corrosion protection.

For the left wall scour protection, three potential solutions were considered as follows:

- Excavate the overburden of the left wall to rock (approximately 50-ft) and provide concrete paving or derrick stone as scour protection similar to the right wall.
- Place a 75-ft width of concrete pavement on the overburden adjacent to the top of the wall and tie it into a 12-ft tall T-wall set back from the wall face 75-ft directing overtopping flows to the downstream end of the training wall.
- Provide armoring over the entire overburden surface and up the slope of the adjacent embankment to above high tailwater.

The solution being provided in this study is to excavate the overburden and construct concrete scour protection over the rock at the heel of the wall. Specifically, concrete will cover all rock adjacent to monolith 93 to monolith 100 of the spillway training wall and will extend 75-ft from the heel of the wall. As with the right wall, passive anchors required to improve flotation stability with Class I corrosion protection spaced at 10-ft on center will be provided for the concrete scour protection. After construction of the scour protection, overburden will be replaced behind the wall, but will be considered sacrificial. The other two solutions listed are considered opportunities to be investigated in PED.

Note: Mitigation, replacement of the fishing pier and recreation area on the left bank, is required in this area. Reference section 6.9 for additional details. The selected solution for scour protection will need to be considered with mitigation requirements and adjusted appropriately.

6.2.6. Stilling Weir
Stability analyses have indicated anchors are required to improve sliding stability for the stilling weir. The plan is to install anchors for each weir monolith. These anchors will be active and will have Class I corrosion protection in accordance with PTI.

Temporary modifications to the stilling weir has been included for construction access. The current design shows partial removal of two weir monoliths on each half of the basin to allow trucks and equipment more efficient access into the spillway stilling basin reducing required ramp heights.

Another design feature being considered with the weir is to permanently remove or partially remove the stop logs of one sluice on each half of the basin to facilitate unwatering each half of the basin post construction for inspection, drain cleaning, and future maintenance/repair will also be developed in PED.

6.2.7. Second Stage Stilling Basin Apron
Assessment of the second stage apron has indicated that it is both unstable and structurally inadequate for the anticipated design discharges. Anchors are required to ensure stability of the apron. In addition, both the baffles and end sill of the apron need removed and replaced with adequately reinforced features of the same size and shape. Therefore, the plan is to remove all baffles and end sill, place adequate reinforcement, and replace with concrete of sufficient strength to account for anticipated hydrodynamic and impact loads.

For the overall apron stability, the design involves the installation of five rows of anchors to improve flotation and sliding stability. Spacing for these anchors will vary as required to avoid baffles, baffle
reinforcement and monolith joints. Also, these anchors will be active with Class I corrosion protection in accordance with PTI. There is potential that the top portion of the apron concrete may need to be removed and replaced similar to the plan for the existing spillway apron. This evaluation will be completed during PED.

6.2.8. Foundation Drainage
An existing drainage gallery is located in the heel of the dam. Due to the installation of anchors, drain efficiency for the existing gallery has been reduced. For the spillway monoliths in particular, it is estimated that at the end of the Phase 4 anchor installation drain efficiency will have decreased from 50% to as low as 16% for some of the monoliths (see Appendix A of the Bluestone Phase 4 DDR for calculations of drain efficiency reductions). Given this reduction in drain efficiency with the extended uplift loading associated with RMP 6, it has been determined that the re-establishment of 50% (or better) drain efficiency is required. To re-establish this efficiency, drainage will be provided through a system that includes strip drains open to the top of rock surface, multiple rows of high angle drains in the basin and along the training walls, and a piped collector system. An option is to implement Phase 6, a new drainage gallery through the toe of the spillway section, to provide additional uplift relief, if needed.

In accordance with EM 1110-2-2200, 50% drain efficiency has been assumed for the new drainage system for design of RMP 6. Instrumentation will be incorporated to monitor uplift pressures after construction (see Appendix F). Also, as recommended by the panel of consultants, a re-evaluation of uplift pressures for the post-anchored condition will be performed at normal pool and flood pool conditions to evaluate impacts of anchor grouting to rock mass permeability and to determine if uplift pressures have been adequately reduced or if there is a need for any additional drainage features or a modification to the current approximate 10-year drain maintenance cycle (Trojanowski, Nuss, & Hertel, 2016). If design assumptions have not been met, a new drainage gallery through the toe of the spillway section may be constructed as a separate phase (Phase 6) to provide additional uplift relief. Information on the geology (such as joint spacing and orientation) that is collected during construction in the basin can be used to optimize the orientation and spacing of drains to maximize the gallery’s effectiveness.

6.2.8.1. Basin Drainage
The new underdrain system serves to reduce uplift for the entire spillway and stilling basin system. Within the stilling basin, a line of high angle drains will be drilled through the existing apron and through the baffle monoliths downstream of the ramps, and strip drains with drilled drains will be provided along the construction joints of the upper and lower slab monoliths. The high angle drilled drains in the apron and baffle monoliths will be piped together laterally within the concrete to carry flows to the divider wall gallery. The drains in the apron are spaced on 8.125-ft on center to be compatible with the planned anchor spacing. The drains for the baffle monoliths are spaced 9.5-ft on centers which is comparable to typical drain spacing for a traditional drainage curtain in a gallery resulting in two drains located downstream of each ramp. Strip drains in the lower basin will be open to the top of rock surface, filled with gravel, and will include drilled drains spaced 30-ft on centers. Drilled drains will be angled 15 degrees to intercept near-vertical fractures in the foundation and extend into rock to EL 1,340-ft to ensure uplift is relieved at the critical failure plane elevation of EL 1,350-ft. The strip drains will also include a network of slotted pipe to transmit water to the divider wall gallery.

A line of drains will also be incorporated along each training wall, angled to intercept high angle fractures in the foundation. These drains are included specifically to intercept fractures that pass under the stilling basin but do not pass under the main dam where they could be intercepted by drains in the inspection gallery (or optional toe gallery, discussed in Section 6.2.8.3). Drains are spaced approximately 9-ft on centers. Drains beneath the right training wall are angled under the wall (to the left) at 15 degrees, and
angled at 36 degrees along the left training wall matching the slope of its landside face to intercept high angle fractures.

### 6.2.8.2. Drain Maintenance

For maintenance, access will be provided to the piping system from the divider wall gallery so that the slotted pipe can be cleaned by high pressure water jetting. Drain covers, if deemed acceptable, will allow access for reaming of drains in the baffle monoliths and training walls where damage by cavitation, high pressure fluctuations and abrasion is less likely since these are generally areas of lower pressures and reduced flow velocity. Drain covers were originally considered, but have been eliminated from the current plan in the upstream and downstream slab monoliths because the potential for damage to the covers and concrete is possibly greater. Design of the drain covers will be further evaluated for robustness, long term performance and operability issues, and constructability in PED, and data from additional physical modeling (separate from Phase B modeling) will be used to evaluate the appropriateness of all locations where they might be included as part of the design. Additional details regarding drain covers can be found in Appendix B.

### 6.2.8.3. Toe Drainage Gallery (Phase 6)

Based on the best available information and conservative assumptions, the anticipated performance of RMP 6 with the underdrain system only (i.e., no toe gallery) is estimated to achieve tolerable levels of risk. However, there is substantial uncertainty associated with this conclusion. The information needed to confirm these findings will be obtained only when a substantial portion of the new stilling basin is put in service and data regarding uplift pressures during pool loading scenarios is acquired. The observation period to acquire this data will begin when the right half of the stilling basin is put in service and is estimated to continue until a date two years after completion of Phase 5.

A re-evaluation of uplift pressures will be performed to evaluate impacts of anchor grouting to rock mass permeability and to determine if uplift pressures have been adequately reduced or if there is a need for any additional drainage features or a modification to the current 10-year drain maintenance cycle. If design assumptions have not been met, a new drainage gallery through the toe of the spillway section may be installed to provide additional uplift relief.

The specific plan for the new toe gallery is to tunnel the new gallery through the spillway toe with rough dimensions of 9.75-ft tall and 6-ft wide. The roof of the new gallery will be semi-circular shaped and a new concrete floor with gutter will be cast in the rough gallery floor with the walls and roof remaining as tunneled. The downstream wall of the new gallery will be located nearly 32-ft from the spillway toe maximizing drain location for uplift relief and minimizing distance between the gallery roof and the spillway anchors. The finished floor of the new gallery will be EL 1,370-ft (five feet lower than the existing gallery). Drains will be drilled into rock and will be spaced at 10-ft on center with approximately 80 total drains being installed. Drains will be angled upstream 15 degrees and drilled into rock to the same elevation as the original drain curtain, EL 1,295-ft. In order to construct the gallery, an access hole will be tunneled on the downstream face of monolith 24 and the tunneling operation will proceed toward the left abutment. After construction and removal of tunneling equipment, the hole will be closed with a new concrete plug. A small electric road header or other means may be used for construction of the toe gallery. Access to the toe gallery will be via new access galleries tunneled to connect to the existing inspection gallery. The toe gallery will tie into the tunneled connection from the divider wall gallery. This connection and two other permanent connections to the inspection gallery may have to be hand excavated, but when possible, equipment may be used.
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NOTES

1. DRILLED DRAINS J ANGLED 5+ 25' TOWARDS H ILLSIDE) BEHIND L EFT TRAINING W/\J.L W ILL FOL LOW THE SLOPE O f THE WALI AND CONS ISTS O F 6" CASING PIPES WI TH CAPS ANO 3" DRILL ED HOLES TO EL. 1340.0.
   CASING PIPES WILL BE ENC LOSED IN CONCRET E ON BACK or TRAIN ING W ILL BE LOCATED ATOP

2. DRILLED DRAINS J ANGLED 15' TOWARDS BAS IN) BEHIND R IGH T TRAINING WALL CONS ISTS O F 6" CASING PIPES WI TH CAPS ANO 3" DRILL ED HOLES

3. DRILLED DRAINS J ANGLED 15" DOWNS T R EAM> JUS T UPS T REA~ OF WEIR CONS ISTS O F 6" CAS ING PI PES WI TH CAPS ANO 3" DRIL LED HOLES TO EL. 1340.0. CASING PIPES WI L T TERMIN ATE 6" B ELO W TOP OF BASIN FLOOR ANO WI L T BE CAPPED, FOR ACCESS, FOR FUTU RE C LEANING.

4. DRILLED DRAINS (ANGLED 15• UPSTREAM) IN UPSTREAM HID DOWNSTREAM MONO LI THS CONS ISTS O F 6" CAS ING P IPES WI TH CAPS ANO 3" DRIL L ED HOLES TO EL. 1340.0. CASING PIP E WILL TERMINATE 6" B ELO W TOP OF BAS IN FLOOR ANO WI L T BE CAPPED, FOR ACCESS, FOR FUTU RE C LEAIN G.

5. DRILLED DRAINS J ANGLED 15' VPS T R EN,l l J~ ST DO WN S T RE ~ OF B AFFLE RAM PS CONSIS T S O F 6" CASING PIPES WI TH CAPS ANO 3 " DR I LLED HOLES TO EL. 1340.0. ACCESS C-'P S, FOR FUT URE C LEANING, W ILL BE A TOP BAS IN FLOOR A T EL. 1 368.0 .

6. 0 DRAIN S (ANGLED 1 5• UP STRE Alt.ll THRU EX ISTING ,C S OF 6" CAS ING PIP ES W IT H CAPS ANO 3 " DRILL ED H OLE S TO EL.0. CAS ING PIPE WILL TERMINATE 6" B ELO W TOP OF NEW CE AND WILL BE CAPPED, FOR ACC ES S, F OR F UTU RE C LEAN IN G.

7. A RE • EVALUATION OF UPL IFT PR ESSURES FOR THE P OSTM-ANCHORED CONDI TIO N W ILL BE PERFOR t.lED AT NORMAL POO L ANO F L OOD POO L CONDI TIO N S OF POSS IBLE, FO R POO L E L. 1450 APPROX IMATEL Yl TO EV ALUATE IMP ACTS TO RO CK MASS PERMEABILIT Y ANO TO DETE RM INE IF UP L IFT PRESSURES HAYE BEEN AIDEQUA T EL Y REDUC ED OR IF T HE RE IS A NEED FO R AINY ADDI TI ONAL DR AI NAGE FEATURES. IF DESIGN ASSUMP T IONS H AVE NOT BEEN M ET , A NEW DRA INAGE GAL LERY THRU T H E TO E O T THE SPILLWAY SECTION MA Y BE INS T ALLED TO PROV IDE ADDI T IONAL U P LI FT REL IEF. THE TOE G ALLER Y WI LL PROVIDE A C U R T AIN or DRAINS SPACED O N T E N FO OT CE NTE R S, FOR A T O T AL ADDITION O F APPRO XI MATEL Y BO NEW DRAINS. D RAINS WI L T BE ANG LED UPSTREAM 15 DEGREES MID DR ILL ED INTO ROCK TO EL. 1 295.0 .

Figure 6-5: RMP 6 – Drainage Plan
6.2.9. Waterproofing of Dam
Since RMP 6 is not designed for floods that overtop the dam, provisions are being made to prevent flooding of the drainage galleries. If the galleries were to flood, any uplift relief provided by foundation drains exiting into the galleries would be reduced or eliminated affecting the stability of the dam. There is also the potential this flooding could increase uplift beyond theoretical if the head within the dam and galleries exceeded the uplift at the location of the drains. Team members walked the site for potential vulnerabilities that will require waterproofing. Design of solutions to address these vulnerabilities will be assessed during PED. For a preliminary list of waterproofing features, see Appendix C, Major Quantities (C.3).

6.2.10. Dam Anchors
With the completion of the current Phase 4 construction contract for the project and as detailed in the Phase 4 DDR, an estimated 66 additional anchors were identified to be installed as part of the post Phase 4 work. However, as described in the Phase 4 DDR, this number was based on certain design assumptions that have changed as additional work at the project has progressed and as RMP 6 has developed. Because of these changes, revision of the total number of dam anchors needed for the project at the end of Phase 4 may be required. The current plan is to verify the actual number in PED to ensure compatibility with implementation of RMP 6 with anticipation that any changes required will be minimal. The changes in design assumptions / parameters are as follows:

- For the post Phase 4 design, the configuration of RMP 6 was unknown. In order to determine the anchor requirements for the spillway monoliths and its associated existing apron, it was assumed the existing spillway and apron received anchors and that there was a rock wedge downstream of the apron of the spillway with scour of the stilling basin rock being addressed. However, it was also assumed that uplift was relieved to tailwater at the end of the existing apron. A new underdrain system provides additional uplift relief, but uplift pressure is not reduced to tailwater until the toe of the stilling weir apron extending the uplift loading by approximately 280 feet.
- Similar to the changes in uplift for the spillway monoliths, the new basin configuration, anchor design, and hydraulic loading in the basin and over the stilling weir have an influence on the stilling basin’s effect to stabilize the spillway monoliths.
- The construction of the Phase 3 Auxiliary Spillway led to refined evaluation of the site geology and location and extent of weak planes of analyses. These changes in failure plane geometry and extent may influence the number of anchors needed for the dam.
- Additional analysis post Phase 4 DDR has indicated tailwaters anticipated for the dam will be higher than used in the design of the 66 anchors. Although likely not to influence the required number of anchors, this increase will be evaluated in PED.
- Similarly, as identified in the Phase 4 DDR, the rock strengths used in the computations for the abutment monoliths will need to be revised with no changes in the anchor requirements anticipated.
- The PDT plans to ensure the dam anchor designs result in a factor of safety of one for sliding for a design pool of top of dam (EL 1,535-ft) with full uplift (no drain efficiency).

6.2.11. Electrical and Mechanical

6.2.11.1. Existing Inspection Gallery
Piping and conduits in the existing inspection gallery will need to be relocated to allow for access to the new divider wall gallery.
6.2.11.2. Pumping of Galleries
A sump pump will be installed at the weir end of the divider wall gallery and will be gravity discharged or pumped out. If the toe gallery (Phase 6) is installed the seepage from this gallery will be routed to the divider wall sump. In addition, the new galleries will include lighting, receptacles and ventilation.

6.2.11.3. Remote Operation of Crest Gates
Remote control of the 21 crest gates will be installed using Programmable Logic Control (PLC). The remote control of the crest gates is required to manipulate the crest gates from a location off the dam in the event that the dam becomes unsafe for project personnel. This is in part due to RMP 6 not being designed for the full IDF and the potential for the dam to be overtopped. Remote operation will be located in two separate locations. One location will be in Pylon 2 and the second location may be at the Original Construction Overlook in a new PLC Building.

6.2.11.4. High Mast Lighting
High mast poles will be installed to have general area lighting on both the right and left abutments of the dam.

6.2.11.5. Multi-Use Recreation Areas on Left and Right Abutment
Electrical and mechanical will support the features installed for this recreational area including electricity, plumbing, sanitation and HVAC. The replacement of the recreational facilities are required as part of the mitigation identified in the SFEIS. A description of these facilities can be found in Section 6.2.12.2 and in the SFEIS.

6.2.12. Mitigation Features
USACE has prepared conceptual mitigation plans and coordinated with the resource agencies to further optimize the conceptual mitigation plans to demonstrate their feasibility. The mitigation concepts were developed by LRH, in coordination with resource agencies including the U.S. Fish and Wildlife Service, U.S. Environmental Protection Agency, National Park Service, West Virginia Department of Natural Resources, West Virginia Department of Environmental Protection, West Virginia State Historic Preservation Office, Virginia Department of Game and Inland Fisheries, and Virginia Department of Environmental Quality. Some of the possible mitigation candidate sites were suggested by resource agencies, while others were identified by USACE. USACE will continue to coordinate with resource agencies as the project moves into PED and the mitigation plans are finalized and implemented. Most mitigation construction is planned to be implemented prior to the start of or concurrently with Phase 5 construction.

6.2.12.1. Aquatic Mitigation
The USFWS Draft Fish and Wildlife Coordination Act Report (FWCAR) classified select habitats in Reconnaissance Area 1 as Resource Category 1 habitat, according to the USFWS 1981 Mitigation Policy. The Final FWCAR updated this determination based on the updated USFWS 2016 Mitigation Policy, classifying the New River, Bluestone River, and their associated aquatic, wetland, and riparian habitats, including the tailwaters of the Bluestone Dam as “high-value habitat”. The USFWS policy for high-value habitat is to recommend avoidance of all impacts, but because alternatives are not available to avoid these impacts for the Bluestone Dam Safety project, the USFWS has decided to seek a net gain in conservation as an outcome on this project. The preliminary Habitat Evaluation Procedure (HEP) conducted by USFWS in 2013 was updated in 2016 to account for the latest design information, proposed minimization measures, and refinement of habitat suitability calculations to determine the number of habitat units impacted by the TSP. In addition to the impact minimization efforts and restoration of directly impacted fish habitat, the USFWS recommended off-site mitigation for the nine aquatic Habitat Units (HUs) impacted at a site that meets at least three of the following eight criteria:
• The site should be adjacent to the New River (river front property). The site can either have intact riparian buffers, and receive mitigation credit for preservation, or lack riparian buffers and receive credit for restoration.
• The site should contain direct tributaries to the New River that are in need of restoration or enhancement. Restoration work can include, but is not limited to, livestock fencing, stream restoration work, enhancement of riparian buffer to reduce erosion (tree/shrub planting), and/or removal of barriers to fish passage.
• The site should be significantly forested or have the potential to be replanted to improve riparian buffers.
• There is the ability to secure the mineral and development rights for the site to ensure that it will not be developed in the future.
• The site should be adjacent to another conservation area (e.g., Wildlife Management Area, State Park, or federally protected land).
• Multiple sites can be selected to reach the mitigation goal, as long as the HU’s generated by each site add up to the amount needed to achieve the goal (9 aquatic HU’s).
• Mitigation sites should have an interested long-term steward that has experience with successfully managing properties for fish and wildlife conservation purposes. USFWS does not expect USACE to manage the site(s) in perpetuity.
• The selected site(s) should be able to achieve the mitigation goal within the lifetime of project construction (8 to 10 years).

USACE has prepared conceptual mitigation plans and performed HEP analyses for each of the proposed candidate sites and coordinated with the resource agencies to further optimize the conceptual mitigation plans to demonstrate their feasibility. The mitigation concepts were developed by LRH, in coordination with resource agencies including the U.S. Fish and Wildlife Service, U.S. Environmental Protection Agency, National Park Service, West Virginia Department of Natural Resources, West Virginia Department of Environmental Protection, West Virginia State Historic Preservation Office, Virginia Department of Game and Inland Fisheries, and Virginia Department of Environmental Quality. Some of the possible mitigation candidate sites were suggested by resource agencies, while others were identified by USACE. These mitigation concepts are individual measures which could be used as stand-alone projects or in combination to provide mitigation for aquatic impacts anticipated from the proposed project.

Conceptual plans for each of the sites were developed based on observations in the field and through desktop review of available site data. Once conceptual plans were developed for each site, preliminary HEP models were utilized to determine the estimated baseline habitat value of each site and estimated mitigation benefit of the possible restoration and/or preservation at each site. Due to the conceptual stage of design for the projects, these HEP models used data gathered during site visits and through desktop review of available data; no formal data plots were established and utilized at the terrestrial sites. Data points were, however, collected at the aquatic sites. Additional HEP models will be run using data points during the PED phase for selected mitigation measures. For the two aquatic sites, Boulder Reefs and Barker’s Bottom, HUs were estimated for the smallmouth bass under baseline conditions and after construction of mitigation features. For the two riparian sites, HUs were estimated for the mink under baseline conditions and ten years after replanting of the riparian area. For these two riparian sites, USFWS also recommended that USACE estimate baseline aquatic habitat units for the smallmouth bass in the portion of river adjacent to these riparian sites to demonstrate the aquatic benefits associated with riparian restoration. The outcomes of the HEPs for the conceptual mitigation plans and additional details on these conceptual mitigation plans can be found in Appendix M of the SFEIS.
The conceptual plans provide an overview of baseline site conditions for each of the candidate sites, proposed ecological enhancements or preservation, success criteria, monitoring plans, adaptive management plans and results of the HEP analyses. The preliminary HEP evaluations completed by USACE demonstrate that sufficient, achievable opportunities are available to adequately mitigate for the anticipated loss of nine aquatic HUs from this proposed project. Additionally, reseeding of disturbed terrestrial habitat within the Construction Work Limits is adequate to restore the 0.79 terrestrial HUs lost during construction.

Once a measure or combination of measures is chosen for implementation of aquatic mitigation, the success criteria, monitoring plans, and adaptive management plans will be further refined in coordination with the U.S. Fish and Wildlife Service to ensure that the selected plan fully mitigates for the habitat lost due to construction of the proposed Bluestone Dam Safety Modification Phase 5. It is anticipated a Supplemental Environmental Assessment would be prepared documenting the mitigation. The mitigation measures are proposed to be implemented prior to or no later than the start of construction of the TSP. The final mitigation plan can be found in Chapter 7 of the SFEIS (Appendix K).

Additional detail and breakdown of cost associated with the aquatic mitigation component of the TSP is presented in Table 6-2 and Table 6-6. Please note the cost estimate for the aquatic mitigation is based off these conceptual mitigation plans presented in Appendix M of the SFEIS. Both the mitigation sites/plans and cost estimate will be revised during the design (PED) phase. The uncertainty of these conceptual mitigation plans are captured in the contingency of the detailed cost estimate.

### Table 6-2: Aquatic Mitigation Costs

<table>
<thead>
<tr>
<th>Environmental Mitigation</th>
<th>Estimated Cost FY2017 Price Level</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>04 – DAMS</strong></td>
<td></td>
</tr>
<tr>
<td>Barkers Bottom</td>
<td>$1.5 M</td>
</tr>
<tr>
<td>Artificial Reefs</td>
<td>$1.7 M</td>
</tr>
<tr>
<td>Lovern Site</td>
<td>$0.8 M</td>
</tr>
<tr>
<td>Contingency</td>
<td>$1.07 M</td>
</tr>
<tr>
<td><strong>30 – PLANNING, ENGINEERING &amp; DESIGN (PED)</strong></td>
<td>$4.1 M</td>
</tr>
<tr>
<td>PED</td>
<td>$3.3 M</td>
</tr>
<tr>
<td>Contingency</td>
<td>$0.9 M</td>
</tr>
<tr>
<td><strong>31 – CONSTRUCTION MANAGEMENT</strong></td>
<td>$0.4 M</td>
</tr>
<tr>
<td>Construction Management</td>
<td>$0.3 M</td>
</tr>
<tr>
<td>Contingency</td>
<td>$0.08 M</td>
</tr>
<tr>
<td><strong>TOTAL ENVIRONMENTAL MITIGATION</strong></td>
<td>$9.6 M</td>
</tr>
</tbody>
</table>

#### 6.2.12.2. Recreational Mitigation

USACE is committed to replacing the existing ADA-accessible public fishing pier in the same approximate location when the TSP and DSA construction is complete. In addition, mitigation for the removal of the existing tailwater fishing pier and the loss of fishing and river access near the dam would include the construction of a replacement access point for fishing downstream of the dam. Several alternative locations on both left and right side banks and configurations of such a replacement fishing pier are currently under consideration were considered. USACE has determined that a location downstream of
the dam on the right descending bank for the replacement pier would be one or more of these alternative locations and configurations would be implemented to mitigate for the loss of fishing access due to construction. This site would be located immediately adjacent to the existing playground on the right descending bank and extend along the riverbank from a point near the existing boat slide and access steps downstream to a point nearing the government property line. The public access facility will be ADA accessible and provide river/fishing access. Direct water access will be provided by steps into the river for wading fisherman and additional fishing positions will be included as platforms (steel grating) extending over the river flow from which both disabled and ambulatory users can fish. The ADA access ramp (steel grating) would be located on the riverbank using a limited footprint (concrete columns) to reduce construction impacts and extended platforms. The designed width of access ramps and platforms will allow safe access by wheelchairs and ambulatory users. The final riverbank surface would likely require slope protection beneath the steel access ramp and land side of extended platforms. Appropriate safety features will be included in the design to avoid accidental falls into the river, but designed so as to accommodate needs of disabled fishermen. A security fence and plantings will be included along the top of the riverbank to separate the existing playground from the new access facility.

USACE would also consider additional opportunities upstream of the dam to provide additional access to the water to mitigate for significant losses to recreation within the downstream areas. This site would be located at the existing Bluestone State Park Boat Launching site. This site is formed on a point created by two small, incoming tributaries into Bluestone Lake. Prior to being burned (vandalism), there was a wooden fisherman access platform (deck) extending from this point into the lake that was ADA accessible and founded on wooden posts with concrete supporting columns placed in the lake. The access road into the boat launching site is very steep, but levels out into a larger paved area at the bottom of the slope. From there the boat launching ramp is located to the right on a steep incline into the water (into one of the two flanking tributaries). Although this site provides access to the lake, its current design as a public boat launching ramp is lacking in boat trailer parking and adequate turning radii for boat trailers. The proposed mitigation ADA access facility would be composed of two or three platforms extending into the lake around the point with each one connected to the next by an ADA designed ramp so that fishing could take place during multiple lake levels while providing adequate space for multiple users. Appropriate safety features would be included in the design to reduce accidental falls into the lake while providing fishing access to disabled users. The platforms would likely be founded on concrete columns to reduce the impact on the lake shoreline and the lake.

Due to significant impacts to the downstream recreational areas on both the left and right descending banks, mitigation would be implemented to restore these areas. Recreation facilities in Bellepoint Park that were removed for staging areas will be restored to improved conditions in order to restore recreational uses after construction is complete. The left abutment recreation area that was removed to allow space for a construction laydown area for Phase 2B would be restored. The features that were impacted include removal of a comfort station, lift station, picnic shelters, playground equipment, and parking lot. Portions of the Route 20 overlook will be completed including additional parking spaces, barriers, walkways, and the existing drainage.
Figure 6-6: Dam Recreation Area Photograph from June 6, 1992 of Left Descending Bank

Figure 6-7: Dam Recreation Area Photograph from April 19, 2016 of Left Descending Bank
6.2.12.3. Vegetation Monitoring
USACE will install vegetation monitoring transects within the inundation zone to determine if potential impacts to vegetation and/or wetland habitat are occurring due to implementation of the TSP and the resulting increased frequency, duration and elevation of upstream inundation. These transects would be positioned to monitor vegetation from elevations immediately adjacent to the river up to the maximum pool elevation of 1523. Data collection at these transects would occur prior to the start of construction, and then at appropriately timed intervals throughout construction or after major pool fluctuations to monitor changes, if any, to species composition and occurrence of invasive species. Sampling locations will include at least one transect through Crumps Bottom wetlands and at least one transect through the bank of the designated Bluestone National Scenic River. If review of monitoring data and consultation with resource agencies demonstrates that any observed change in species composition exceeds the parameters of natural succession and is attributable to the impact of the TSP, appropriate mitigation measures would be implemented.

6.2.13. Non-Structural Measures
As part of RMP 6, improved risk communication is an essential non-structural measure of the plan. Regular communication with the public regarding the flood risk associated with living downstream of a dam will aid in reducing consequences and serve as a reminder that risk is a shared responsibility between the public, state and local officials, emergency managers and USACE (dam owner). Improved risk communication will utilize various techniques, methods and tools. Some examples include: social media, traditional advertisement, public meetings, public service announcements, press releases, regular communication with local officials and emergency managers, signage, pamphlets, booths at local fairs and festivals, and development of consistent messaging for use by dam and District personnel.

As stated in Section 3.2.3.8 and 3.2.3.9, improved warning systems and evacuations plans will be implemented by the local entities and the study assumes these will be implemented whether the recommended plan is constructed or not. Additional investments to improved warning systems above and beyond what the locals are anticipated to implement was determined to provide negligible risk reduction; therefore, additional improvements were not considered in development of alternatives.

6.3. Implementation and Construction of RMP 6
RMP 6 will be implemented as quickly as funding and legal constraints allow. The contract acquisition strategy and cost estimate assumes two single, large procurements for the major construction work, one for Phase 5 and one for Phase 6. These contracts are assumed to be a request for proposal (RFP) due to specialized construction techniques associated with directional drilling, anchor installation, tunneling for galleries, and risk associated with removal of concrete at the toe of the dam.

6.3.1. Construction Work Limits (CWL)
Construction work limits have been defined along with applicable restrictions. There is limited land on Government property for construction use. The dam and New River sit in a very deep valley. The city of Hinton, West Virginia is immediately downstream of the dam and Government Property Boundary. In spite of these challenges there have been many large construction contracts awarded and administered during the ongoing DSA.

The Construction Work Limits are delineated in Figure 6-8 and in drawing 01 in Appendix C. This includes construction area limits as well as 10 areas suitable for Laydown and staging areas. All work limits are within the Government Property Boundaries except for a 3 acre site (AREA 1) on the left side of the New River downstream of the dam. This land will be leased, prior to construction contract being awarded, for
use by the contractor. Several previous contractors as well as current contractors have and are utilizing this area for laydown, staging, material storage, material processing, and stockpiling.

The construction work limits have been laid out for the least impact to the environment and providing the contractor the most suitable adequate room to construct the features.
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A
ILEA TH ERWOOD LANDING
BOAT RAMP
r UPSTREAM
---
BLUESTONE LAKE
LAYDOWN AND STAGING AREAS
AREA 1-APPROXIMATELY 1 ACRE SITE DOWNSTREAM
OF THE DAM OUTSIDE THE GOVERNMENT PROPER T Y
BOUNDARY. LAND TO BE LEASED BY THE
GOVERNMENT PRIOR TO AWARDING A CONSTRUCTION
CONTRACT. SITE IS SUITABLE FOR LAYDOWN, STAGING,
MATERIAL STORAGE, MATERIAL PROCESSING,
STOCKPILING AND CONCRETE BATCH PLANT.
OFFERS DIRECT ACCESS TO THE CONSTRUCTION SITE
WITHOUT HAVING TO TRAVEL ON RT. 20.

AREA 2-APPROXIMATELY 3/4 ACRE RELATIVELY FLAT
SITE ADJACENT TO THE BASIN ON THE LEFT SIDE.
THE SITE IS SUITABLE FOR LAYDOWN, STAGING,
MATERIAL STORAGE AND STOCKPILING. SEE NOTE 3.

AREA 3-APPROXIMATELY 5,000 SF AREA ADJACENT
TO RT. 20 ON THE LEFT MUYMENT. SITE IS
SUITABLE FOR A SMALL STAGING AREA AND ALSO PROVIDES
ACCESS TO TOP OF DAM.

AREA 4-APPROXIMATELY 1/4 ACRE SITE JUST SOUTH
OF THE RESIDENT ENGINEERS OFFICE. SITE IS
SUITABLE FOR A SMALL STAGING AREA, MATERIAL
STORAGE AND STOCKPILING.

AREA 5-APPROXIMATELY 3/4 ACRE RELATIVELY FLAT
SITE LOCATED DOWNSTREAM OF THE
RIVER ON THE RIGHT SIDE OF THE NEW RIVER
AT THE BELLE POINT AREA. SITE IS SUITABLE FOR
LAYDOWN, STAGING, MATERIAL STORAGE, MATERIAl,
PROCESSING, STOCKPILING AND CONCRETE BATCH PLANT.
SEE NOTE 5 THIS SHEET.

AREA 6-APPROXIMATELY 3.5 ACRE RELATIVEFLY FLAT
SITE LOCATED DOWNSTREAM OF THE DAM ON THE
RIGHT SIDE OF THE NEW RIVER. SITE IS SUITABLE
FOR LAYDOWN, STAGING, MATERIAL STORAGE,
MATERIAL PROCESSING, STOCKPILING AND CONCRETE BATCH
PLANT. THERE IS ALSO A 4,800 SF PRE-ENGINEERED
STEEL STORAGE BUILDING ON THE SITE.

AREA 7-APPROXIMATELY 3/4 ACRE SITE ADJACENT
TO THE SOUTH SIDE OF THE EXISTING OPERATIONS
MAINTENANCE BUILDING. SITE IS SUITABLE FOR
STORAGE.

AREA 8-APPROXIMATELY 1 1/3 ACRE RELATIVELY FLAT
SITE LOCATED ADJACENT TO THE PENSTOCK STAGING
BASIN ON THE RIGHT SIDE. SITE IS SUITABLE FOR
LAYDOWN, STAGING, MATERIAL STORAGE, MATERIAl,
PROCESSING, AND STOCKPILING.

AREA 9-APPROXIMATELY 1 1/3 ACRE RELATIVELY FLAT
SITE LOCATED ADJACENT TO THE PENSTOCK STAGING
BASIN ON THE RIGHT SIDE. SITE IS SUITABLE FOR
LAYDOWN, STAGING, MATERIAL STORAGE, MATERIAl,
PROCESSING, AND STOCKPILING.

AREA 10-A NARROW STRIP OF CLEARED LAND ON
EACH SIDE OF THE ACCESS ROAD LEADING TO THE
LEATHERWOOD BOAT LANDING.

OVERALL SITE PLAN
SCALE IN FEET

NOTES
1. UPSTREAM WORK AREAS PROVIDE MARINE ACCESS TO THE
UPSTREAM END OF THE DAM.
2. FOR LARGER SCALE OF SHORE FEATURES SEE SHEETS 02 AND 03.
3. AFTER THE CONSTRUCTION FEATURES IN THIS AREA HAVE BEEN
COMPLETED THE AREA WILL BE UTILIZED AS A RESTORED OPERATIONS
AREA. A NEW COMFORT STATION AND RECREATION AREA WILL BE
CONSTRUCTED. A NEW HANDICAPPED ACCESSIBLE FISHING PIER, MUCH
LIKE THE EXISTING PIER, WILL BE CONSTRUCTED.
4. STRAND ANCHORS WILL BE INSTALLED IN VARIOUS LOKALS THROUGHOUT THE
RESERVOIR. SEE STRUCTURAL APPENDIX.
5. DURING THE POND STAGE OF THE PROJECT AREA MAY BE CONSIDERED
AS A PERMANENT DISPOSAL SITE FOR CLEAN EXCAVATED MATERIAL.
6. Figure 6-8: Work Limits Topographic Mapping
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6.3.2. Laydown and Staging Areas
There are several areas within the contractors work limits that are suitable for Laydown and Staging. Ten (10) have been identified ranging in size from a few thousand square feet to 3.5 acres. These areas are described in the following sections and illustrated in Figure 6-8. All of these areas, with the exception of Area 1 are located on Government property. For further details, see Appendix C.

6.3.3. Disposal Areas
Excavation of bedrock from the stilling basin floor, overburden behind the left training wall, and demolition of existing features (e.g. concrete) will generate considerable quantities of material for disposal. The majority would be clean rock (e.g. free of soil). Any construction material will be disposed of in a licensed landfill or other appropriate location in compliance with applicable laws. In terms of clean material, there is limited space onsite for disposal of material, and as a result, alternate locations will be evaluated to reduce quantity of material disposed in a licensed landfill (40+ miles away from site). One potential option is to place the material adjacent to or against the right downstream abutment/hillside. Alternatively, the contractor may choose to sell clean material in compliance with all applicable laws. Temporary stockpile locations used for Phase 3 construction are potential locations. To reduce risk and uncertainty with regards to project costs, the designated government disposal sites will be secured prior to contract execution.

Several potential alternative offsite locations have been identified to accommodate the large amount of material (100,000 to 250,000 CY) to be removed during project construction. These areas are illustrated and outlined in the draft MFR, Bluestone Dam, Dam Safety Modification Study and Environmental Impact Statement-Potential disposal sites for Dam Safety Modification Construction (Appendix K).

Further investigation is planned during the PED phase to identify alternative sites prior to contract award. See Appendix C for additional discussion and site plan drawings as well as Section 6.4.4.

6.3.4. Material Investigations and Availability
Material availability is a potential issue; however, the concrete materials for these efforts will be very similar to those utilized for the Phase 3 construction contract. Lessons learned from this contract are being applied to the cost and construction assumptions for RMP 6. Therefore the specifications will address the potential for specific concrete constituents. The Contractor will be required to select and test the proposed materials and test and fully vet alternative sources as a safeguard. It is anticipated that this will reduce the risk of a production shutdown. Additional revisions will be made to the specifications to expand alternatives and reduce the risks associated with material availability. The materials that may be affected are described in the following paragraphs.

6.3.4.1. Portland Cement
It is difficult to get cement other than general purpose Type I and Type I/II. Low alkali cement is also becoming increasingly difficult to obtain. Specialty cements like Type II (LA), Type II (MH) and Type II (MH) (LA) are in short supply and high demand. The use of Type I/II with fly ash should be able to compensate however the fly ash content may need to be raised. Concrete with a 50/50 blend of Type I/II and low calcium fly ash have been successfully used on other projects. The higher fly ash content will help to control the alkali-aggregate reactivity in lieu of low alkali cement, and provide the thermal control in lieu of type II (MH) cement. It is important that the proposed concrete mixture proportions be tested in accordance with ASTM C1567 and C1105 to assure reactivity is reduced to acceptable parameters. Limits will be set and thermal testing will be required to assure that the heat of hydration is reasonably controlled.
6.3.4.2. Class F Fly Ash
It must also be recognized that fly ash sources are diminishing as plants convert to lower cost natural gas. Electricity producers have also shutdown older coal-fired plants that cannot be economically upgraded to comply with new coal burning regulations. As a result, the remaining fly ash sources are in high demand, and shortages are becoming more prevalent in the spring and fall when plants perform maintenance in preparation of the peak demands of summer and winter. It is anticipated that this can be addressed by having a fully tested and vetted alternative source available. In addition, Contractors may need to consider having the ability to stockpile fly ash to assure availability through production.

6.3.4.3. Ground-Granulated Blast Furnace Slag (GGBFS)
GGBFS can be incorporated into concrete mixture proportion to address alkali-aggregate reactivity and thermal control. GGBFS currently is being used to offset the cost and availability of cement therefore it is ground to react faster and does not offer the thermal benefits that a coarser GGBFS offers. However there may be some benefit to allowing ternary mixtures using cement, fly ash and GGBFS to control reactivity and heat.

6.3.4.4. Other Constituents
No availability issues are anticipated with the other concrete constituents. As demonstrated in Phase 3, aggregates, admixtures and suitable water historically have not been an issue and are readily available locally.

6.3.5. Construction Schedule
The construction schedule can be found in Appendix I.

6.3.5.1. Phase 5 Construction Sequencing
The critical path sequence starts with constructing the temporary middle cofferdam. Once complete, unwatering the right side of the first stage stilling basin can occur and construction will begin. For access the contractor could partially demolish one or two monoliths in the weir down to EL 1,378-ft to make access less challenging, but construction of the downstream cofferdam may be necessary for this access. The priority for the contractor will be to tunnel the divider gallery connector and complete the modifications to the existing stilling basin apron prior to excavation for the super baffle monoliths and apron extension.

Existing Apron Sequence of Work:

1. Demolish baffles, end sill, and 5-ft surface demolition of concrete
2. Install drain plumbing along with formwork
3. Place concrete
4. Install anchors
5. Drill drains

Once the existing apron work is complete then the contractor can start excavation. However, there can be some overlap depending on how far away the excavation is from the unfinished apron. Once the existing apron is complete then all the excavation can be completed. The formwork and placement of concrete for the super baffle monoliths and apron extension can begin near the end of excavation. The sequencing of the construction is likely to begin from the center of the stilling basin outwards; therefore, the prioritized work will be the construction of the permanent divider wall. This will also allow for the cofferdam to be removed sooner allowing a smoother transition to the left side of the stilling basin.
Super Baffle Monolith and Apron Extension Sequence of Work:

1. Rock excavation
2. Foundation prep
3. Formwork (concurrent work)
   a. Installations of the drain plumbing
   b. Reinforcing steel
   c. Hole block out for anchors and the vertical portion of the drains
   d. Water stop
4. Place concrete
5. Drill drains

Super Baffle Monoliths and Apron Extension Priority:

1. Priority will be the permanent divider wall construction
2. Baffle and downstream monoliths concurrent construction
3. Upstream monoliths construction

Anchor installation in the new concrete can begin at about 80% to 90% completion of the concrete work. Installing anchors is the last step in completing the right side of the basin, once complete and equipment is removed from the basin transitioning to the left side can begin. At this point the permanent divider wall will be in place and then the unwatering of the left side can begin quickly. Once the left side is unwatered the sequence work performed on the right side will be repeated.

The non-critical path sequence of work items include the training wall scour protection, anchors, and drains; the weir anchors; second stage anchors; and baffle and endsill reconstruction. The left training wall excavation can begin after the temporary divider wall is in place and the floating platform is removed from the basin. Once the concrete is placed for the left training wall the fill placement can start. It is assumed that the area behind the left training wall can be used as a disposal area for the excavation in the basin, and the rock will be processed to specified gradation on the right side and transported to the left side. Another item that can be constructed is the right training wall work, which appears can be done at any time that contractor feels is appropriate.

**6.3.5.2. Phase 6 Construction Sequencing**

If after the uplift assessment as described in section 6.2.8.3 it is determined that the toe gallery is needed, the procurement process will begin. This work can potentially take several years due to the expected low production rate of 1 to 3 CY per day.

Toe Gallery Sequence of Work:

1. Excavation main tunnel (access via downstream face monolith 24)
2. Excavate connectors to the inspection gallery
3. Place concrete floor with gutter
4. Drill drains
5. Install instrumentation
6. Place concrete plug in access point at monolith 24

**6.3.6. Care and Diversion of Water**

Considering the stilling basin modification will impact the project’s ability to release water, multiple care and diversion options were developed and screened. There are limited options for effective and safe care and diversion of water during the construction activities. During normal operations today, there are two
means to pass flow, one through the sluices and the other over the spillway crest. Both means utilize the stilling basin structure that will be modified under Phase 5 construction. There is also an auxiliary penstock spillway and stilling basin that is available for operation. However, passing flow through this basin will require removal of the existing cofferdam (dike) which provides access for Phase 5 construction. Given the conservation pool is approximately 125-ft below the top of dam, diversion over or adjacent to the dam is not feasible. Given the physical constraints of the project, the following options have been considered and evaluated:

1. Diversion of normal and minor flood flows through half of the existing stilling basin, while the other half is dewatered for construction.
2. Diversion of normal and minor flood flows through two-thirds of the existing stilling basin, while the remaining one-third is dewatered for construction.
3. Diversion of normal and minor flood flows through the auxiliary penstock spillway and stilling basin, while the entire primary stilling basin is dewatered for construction. For this analysis, the installation of four controlled penstock gates was considered.
4. Both diversions through the auxiliary penstock spillway and half of the existing stilling basin, while half of the primary stilling basin is dewatered. For this analysis, two controlled penstock conduits were considered in addition to half of the existing stilling basin.
5. Diversion of normal and minor floods flows through half of the existing stilling basin and two open (uncontrolled) penstocks; while half of the primary stilling basin is dewatered for construction.

The evaluation of the proposed options for the care and diversion of water included the following considerations:

1. Cost associated with each alternative
2. Impact to overall implementation of the risk management measure (functioning stilling basin)
3. Recreation and environmental impacts within the pool due to increase pool loading frequency and duration.
4. Recreation and environmental impacts downstream of the project
5. Impacts on operations during the period of construction

The evaluation of these considerations was primarily done throughout the process of developing the RMPs during the formulation and evaluation process. The feasibility of the various options was evaluated by the PDT. In relation to these evaluations, the following items were considered for each option.

Option 1 – Diversion through half the stilling basin

- Reduces the construction duration over option 2
- Eliminates the need for multiple temporary cofferdams as included in option 2
- Less costly than options 2 and 4
- Does not inhibit operation of a sluice due to cofferdam requirements as with option 3
- Increase pool retention and impact on flood risk management over options 2 and 4
- Does not allow for passage of drift and debris through the drift tower

Option 2 – Diversion through two-thirds of the stilling basin

- Reduces the pool retention over option 1 for a portion of the construction period
- Allows for easier construction of permanent divider wall
• Divider wall may not be used as part of the cofferdam due to sequencing
• Reduces the flood risk during construction over option 1
• Requires multiple temporary cofferdams adding to construction cost
• Eliminates use of the drift tower for approximately one-third of the construction period
• Complicates access to downstream features during the dewatering of the center one-third basin.

Option 3 – Diversion through the auxiliary penstock spillway

• Allows for dewatering of the primary stilling basin without temporary coffer dam
• Results in the shortest construction period and quickest implementation of risk reduction
• May allow for larger release capacity below spillway invert
• Requires the purchase of service gates significantly increasing cost
• Concerned with penstock stilling basin performance for low tail waters
• Penstock operation may induce damage to right descending stream bank
• Remove auxiliary penstock basin cofferdam and provide alternative construction access
• Address issue with invert elevation being at summer pool

Option 4 – Diversion through the controlled penstock stilling basin and half of the primary stilling basin

• Least impact on flood risk management
• Reduced construction duration over option 2
• Highest operational flexibility
• Requires the purchase of service gates significantly increasing cost
• Penstock stilling basin performance for low tail waters needs considered
• Penstock operation may induce damage to right descending stream bank
• Remove auxiliary penstock basin cofferdam and provide alternative construction access
• Address issue with invert elevation being at summer pool

Option 5 – Diversion through the uncontrolled penstock stilling basin and half of the stilling basin

• Least impact on pool retention
• Reduced construction duration over option 2
• Reduction in benefits for flood risk management for duration of construction
• Penstock stilling basin performance for low tail waters needs considered
• Penstock operation may induce damage to right descending stream bank
• Remove auxiliary penstock basin cofferdam and provide alternative construction access

The evaluation of the effect on pool and discharge are illustrated in Figure 6-9 through Figure 6-11. Considering both the qualitative and quantitative information, Option 1 is the preferred because it provides the best balance between costs, risks, and impacts. A conservative estimate of 1-demobilization/damage event during the 10-year construction period is considered appropriate.
Figure 6-9: Care and Diversion of Water - Annual Chance Exceedance

Figure 6-10: Care and Diversion of Water Alternatives - Pool Duration Exceedance
6.3.6.1. Cofferdams
The spillway and stilling basin features will be constructed in two stages. Data collected during physical modeling efforts and the constructability review determined the size, height and material proposed for the cofferdams designs.

The 37-ft high cofferdam was calculated based on a reservoir pool EL 1,520-ft. The annual chance exceedance of the top of flood control pool ranges from approximately 1 in 50 to 1 in 300. At that pool elevation, the maximum discharge through the non-construction side of the dam with the sluices (8) full open is 33,440 cfs, corresponding to an EL 1,401-ft, with 2-ft of freeboard. During the Phase A physical modeling experiments, observations of the flow conditions near the divider wall (which is representative of the cofferdam) indicated that the proximity of the cofferdam in relation to the sluices caused run-up that was not contained with the wall elevation at 1,401-ft. Therefore, additional height was added to the cofferdam for that consideration, and additional refinements will be made during Phase B physical modeling during PED. Refer to Appendix D (H&H) for additional details.

The right side of the basin will be constructed first. To construct the right side a cofferdam, in the basin, perpendicular to the dam, from the dam to the existing weir, will be required. A short length of cofferdam is also required over the weir and downstream second stage basin. A downstream cofferdam, parallel with the dam, is required. It is expected to be a 10-ft tall granular fill and may be used as a causeway for construction traffic. An earthen cofferdam just downstream of the auxiliary penstock stilling basin was used during Phase 3 construction. This cofferdam will remain in place and the proposed downstream cofferdam will tie into its embankment on the right end.

Prior to construction of the downstream cofferdam, the existing stilling weir can provide an adequate damming surface such that work may proceed within the first stage basin. This is due to the crest of the weir (EL 1,391-ft) being well above expected tailwater (< EL 1,376-ft). However, access to the first stage
stilling basin will be limited without the benefit of the downstream cofferdam serving as construction access to this area.

During construction of the right side of the basin, the permanent divider wall will be constructed. This wall will act as the cofferdam through the basin during left side construction. A short length of cofferdam will be constructed over the weir and second stage basin and tie-in to a downstream granular cofferdam as previously described. The cofferdam will tie into the left descending bank.

The existing weir itself acts as a cofferdam for construction of the features upstream of it. However, once the weir is notched for construction access, its damming height will be reduced to EL 1,378-ft. During PED there will be several opportunities to reduce the cost of the cofferdams. Alternative solutions to cellular structures, the post and panels as well as the granular cofferdam features are being investigated and considered.

The removal of the existing auxiliary penstock stilling basin earthen cofferdam, along with an excavated exit channel, was included in the Phase 3 contract. The work was deleted from the Phase 3 contract and the cofferdam will be used as access into the right side construction area. It will be removed after Phase 5 construction is complete. A 10H on 1V penstock exit channel will be excavated, through rock, until it daylights downstream. All this work will be performed in wet conditions.

### Middle (Basin)

The plan for dividing the basin in half until the divider wall is installed is to construct a cellular cofferdam in front of monolith 35 extending from the spillway face to the stilling weir. The cofferdam will be 36-ft in diameter with a top elevation of EL 1,405-ft (design height is EL 1,401-ft with 4-ft freeboard) and a bottom elevation EL 1,368-ft +/- . It will be constructed in the wet. For the construction of the cells PS 27.5 piling, ASTM A328, will be used and the cell fill will be No. 57 aggregate. Due to the irregular founding surface a 2-ft tremie concrete placement is planned for the base of the cell to seal the cofferdam and prevent underseepage and erosion of the fill. Stability analysis was performed for the cofferdam associated with the adjacent divider wall excavation (See Appendix B for the analysis and Appendix E for a discussion of geotechnical aspects). In order to address deep seated sliding of the cofferdam, a thrust block with 14 strand anchors will be required at the toe of the structure. These anchors will be temporary and designed to facilitate easy removal. For the tie-ins to the dam and downstream face of weir, steel pile with concrete panels will be used.

### Downstream (Tailwater)

A temporary granular cofferdam is proposed just downstream of the second stage stilling basin to allow installation of downstream weir and second stage apron anchors in the dry. Due to the relatively low height needed (10 to 13-ft), an embankment was selected in lieu of more costly sheet pile cellular structure. In addition, constructing sheet pile cells, on the uneven surface of the bedrock, would likely require grouting in the base of the cells to reduce seepage. This grout could potentially enter the river, which is environmentally unfavorable. Also, large bags (i.e., Super Sack or Big Bag) with rocks could be used as a more environmentally acceptable method compared to the proposed traditional rockfill cofferdam.

The impervious component of the granular cofferdam will be a PVC geomembrane embedded within the wet side slope. A PVC geomembrane is proposed due to its relatively high friction angle (aggregate to geomembrane interface friction) and puncture resistance when compared to HDPE (see Appendix E, Exhibit 7). The embankment will consist of processed rock taken from the right basin floor excavation or more likely from a commercial source with gradation similar to WVDOH Class No. 7 aggregate (2 to 8-inch diameter stone), and will have 2H:1V out-slopes, 70 to 82-ft basal width (varies depending on the top of
rock elevation), a 30-ft wide crest at EL 1,378-ft, and riprap armoring the wet side. See Figure 6-12 for a typical section.

![Figure 6-12: Typical Section of the Downstream Cofferdam Embankment](image)

For downstream cofferdam construction, the embankment template to the dry side of the geomembrane will be first built out to EL 1,377-ft. The membrane will then be placed in the wet along the top of rock surface for 10-ft and up the 1H:1V riverward embankment face and extended 3-ft back horizontally for key-in. Note that the depth of water is typically only a few feet at this location, and will allow observation of the placement surface for the geomembrane. The remainder of the rockfill (riverward of the geomembrane) will then be placed in lifts by excavator. The weight of the overlying fill will act to seal the geomembrane to the riverbed (bedrock), and the 10-ft extension of membrane along the river bottom will ensure a seal is achieved in the event of an uneven rock surface. The top 1-ft of cofferdam will be surfaced with a dense graded aggregate, and then graded riprap will be placed over the outer face.

The proposed granular cofferdam will start at the downstream end of the right training wall, where it will tie into the existing Phase 3 earthen cofferdam. It will then parallel the dam, just downstream of the second stage apron, for about 450-ft before turning upstream and tying into the cellular portion of the cofferdam for unwatering of the right side of the basin. Following completion of work in the right side of the basin, the cofferdam will be removed. The left side cofferdam will then be built, with placement starting from the left abutment riverbank, just downstream of the left training wall, and working toward the center of the river. This phased approach to spillway unwatering is necessary to maintain flows and normal operation of the dam. It is anticipated that placement of the cofferdam fill prior to geomembrane installation will occur by continually pushing out the crushed rock by dozer until the cofferdam reaches its termination at the weir directly downstream of dam monolith No. 35.

There is relatively little uncertainty in the design or performance of this cofferdam, with the exception of the puncture resistance of the geomembrane to the surrounding rockfill, seepage between the geomembrane and the foundation bedrock, and leakage through the bedrock itself. In regards to the seepage, a defect (sand layer) between the geomembrane and the bedrock was modeled in the seepage analyses used for evaluation and design of the cofferdam. The seepage quantity from the model (0.5 GPM/LF) does not account for leakage from joints or fractures in the bedrock, as these features are difficult to predict (e.g. location and interconnectedness) and model – see Geotechnical Appendix for additional discussion. Leakage beneath the cofferdam is expected and steps are under consideration for controlling/diverting this water during construction. To address the puncture resistance, cushion layers of gravel sized material (e.g. No. 57 aggregate) or geotextiles may be needed on either side of membrane.
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Figure 6-13: Cofferdam Right Side of Basin
Figure 6-14: Cofferdam Left Side of Basin
6.3.6.2. Bypass System for Low Flow Augmentation
A bypass system will be utilized to minimize impacts associated with altered flow regimes during the proposed construction. During low flow conditions (610 cfs) portions of the tailwater area could become dry that would otherwise not occur. Numerical hydraulic modeling indicates this is especially a concern when the downstream right half of the primary basin is un-watered for construction and the dam is releasing water from the left half only. In this condition, a bypass system will be installed that will transmit water from the left half of the stilling basin to the non-operational side of the stilling basin to more evenly disperse water downstream of the non-operational side. Water will be piped from the operational side and will extend downstream of the end sill. The pipe will transmit flow downstream of the cofferdam. The system will include valves that can be opened during low flow events. The numerical hydraulic modeling of this bypass system in place indicates that it should simulate the existing condition during low flow conditions. This design will be further refined during the PED phase.

6.3.7. Temporary Instrumentation during Construction
Instrumentation for the selected plan is divided into two categories; temporary instrumentation during construction and permanent instrumentation post construction. Temporary instrumentation will involve monitoring of the right and left segment construction cofferdams as well as the existing spillway, training wall and weir monoliths. Permanent instrumentation involves the installation of new instruments in the divider wall gallery and spillway for future performance monitoring purposes.

The right segment construction cofferdam will consist of seven temporary 36-ft diameter steel cofferdam cells constructed just west of the proposed stilling basin divider wall, a temporary granular cofferdam located just downstream of the second stage basin monoliths and utilization of seven existing right training wall monoliths.

The left segment construction cofferdam will consist of utilization of the new stilling basin divider wall, two 36-ft diameter steel cofferdam cells constructed just downstream of the new stilling basin divider wall, a temporary granular cofferdam located just downstream of the second stage basin monoliths and utilization of seven existing left training wall monoliths.

The primary monitoring concerns for temporary instrumentation are sliding and deformation of the cofferdam cells; sliding and rotation of the spillway divider and training walls; sliding of the spillway and weir monoliths and phreatic levels in the cofferdam cells and granular cofferdam. An Automated Data Acquisition System (ADAS) will be constructed to monitor for these concerns. The ADAS will be designed around a distributed intelligence network developed to allow localized evaluation of automated instrumentation. Instrument readings will be evaluated against established threshold limits with the ADAS having the capability to execute siren and beacon warnings in the case of threshold exceedance.

Additional details on instrumentation can be found in Appendix F.

6.3.8. Dam Operation during Construction
The maximum flows (or stages) that can be maintained at points along a channel below a dam are called control flows (or stages). The operation of Bluestone Dam is guided by established water level control stages downstream of the dam. When downstream water levels are forecasted to exceed established thresholds, more water is retained in Bluestone Lake. The control stage below Bluestone Lake is 10.7-ft (89,400 cfs) on the U.S. Geological Survey (USGS) gage at Hinton, West Virginia. The control stage at Kanawha Falls is 22-ft (146,000 cfs), and the control stage at Charleston, West Virginia is 36-ft (~150,000 cfs).
cfs, depending on the stage of the Ohio River). The control stages would remain effective during construction of RMP 6.

Between the months April and November, discharge from the dam is regulated to maintain a summer pool of EL 1,410-ft for recreation and fish and wildlife conservation. In the fall, the pool is drawn down to the winter pool at an EL 1,406-ft to allow for additional flood control storage. In order to sustain downstream aquatic populations, a minimum discharge of 610 cfs is always maintained. Given that half of the stilling basin would be closed to flow at any one time during construction of RMP 6, the sluice gate operational scenario during construction would differ from existing operations. However, the target water elevations (summer and winter pools) would still be maintained under non-flood conditions and the necessary flow to obtain these elevations and the minimum discharge would be achieved through the use of up to eight sluice gates at a time.

If a flood event were projected to require the use of more than the eight available sluice gates at one time, several operational scenarios could be employed. First, the construction would be stopped within the dry half of the stilling basin, personnel would be evacuated, and as much equipment and materials as possible would be removed prior to opening the additional eight sluice gates or any and all of the 21 crest gates. However, depending on the progress of construction activities, it may not be safe to open the sluice or crest gates until the existing apron is stabilized. If it was forecasted the flood control pool would be exceeded, flow would be augmented by opening one or more of the penstocks on the right side of the dam. Because the penstocks gates may be severely damaged during use, the open penstocks may not be readily closed requiring design, procurement, and placement of bulkheads after the pool drops below EL 1,410-ft.

In order to meet required downstream flow conditions and water level control stages during construction, out of pool conditions are likely to occur upstream of the dam more often, for longer durations, and at higher elevations. Out of pool refers to higher than normal pool (summer pool is EL 1,410-ft and winter pool is EL 1,406-ft) elevation upstream of the dam. A comparison showing the potential effects between the current operation and water control during construction can be found in Figure 6-9 through Figure 6-11. As these figures demonstrate, exceedance of higher elevations and durations are more likely during construction of RMP 6 with closure of half of the basin. Historically, out of pool conditions have occurred on an average of 18 days per year and may increase to an average of 54 days per year during construction.

A water control manual update(s) will define project outflow restrictions throughout construction. The safe discharge will vary throughout construction based on the reduction in effective weir length (stilling basin split in half) as previously described and incremental completion of cofferdams and portions of the stilling basin modifications. A preliminary plan for updates to the water control manual for construction is detailed in Table 6-3 and represented graphically in Figure 6-15.

It is anticipated that no further changes to the water control will be necessary for Phase 6 construction.
## Table 6-3: Water Control Plan Activities

<table>
<thead>
<tr>
<th>Year</th>
<th>Action**</th>
<th>Trigger</th>
<th>Flood Control Pool EL (ft)</th>
<th>Primary Spillway Discharge</th>
<th>Penstock Discharge</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>2016</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Expire at end of Phase 4 Construction</td>
</tr>
<tr>
<td>2017</td>
<td>Deviation</td>
<td>Risk assessment of operating scenarios</td>
<td>TBD</td>
<td>~140 kcfs</td>
<td>~140kcfs</td>
<td>Risk assessment underway to confirm or amend 2016 deviation as well as add penstock operation.</td>
</tr>
<tr>
<td>2019</td>
<td>Update</td>
<td>Phase 4 Complete</td>
<td>1,520</td>
<td>~140 kcfs</td>
<td>~140kcfs</td>
<td>Flood Control Pool of 1520 restored with Phase 4 anchors complete. This matches the assumption for the FWAC</td>
</tr>
<tr>
<td>2023</td>
<td>Update</td>
<td>Phase 5 Start</td>
<td>1,520</td>
<td>~100 kcfs*</td>
<td>~140kcfs</td>
<td>Right half of basin (8 of 16 sluices) under construction. Discharge includes demobilization of contractor and operating sluices only on right side.</td>
</tr>
<tr>
<td>2028</td>
<td>Update</td>
<td>Right half of basin operational</td>
<td>1,520</td>
<td>~230 kcfs*</td>
<td>~140kcfs</td>
<td>Left half of basin (8 of 16 sluices) under construction. Discharge includes demobilization of contractor and operating sluices only on left side.</td>
</tr>
<tr>
<td>2032</td>
<td>Update</td>
<td>Phase 5 complete</td>
<td>1,520</td>
<td>~840 kcfs</td>
<td>~150kcfs</td>
<td>Discharge for top of dam, EL 1,535-ft. IDF overtops dam by ~18 feet. Penstock operations will be updated.</td>
</tr>
</tbody>
</table>

*To reach this discharge in the primary spillway requires demobilization of the contractor during construction of phase 5 for implementation of RMP 6. This value could vary up or down depending on the state of the stilling basin during construction activities at the time and potential risk imposed by a flood event.

**A deviation is change to the water control plan that is expected to last approximately 3 years or less. Any change to the water control plan greater than 3 years requires an update.
Figure 6-15: Timeline of Water Control Plan Activities
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During construction of RMP 6, the drift and debris tower cannot be used. During low flow events, project personnel will need to revert back to passing drift and debris through the sluices. This creates an increased labor cost for project personnel during construction which would be covered as part of implementation costs.

### 6.3.9. Dam Safety Risk during Construction

As described in Chapter 3 and 4, the risks associated with breach of Bluestone Dam are above tolerable risk guidelines and are primarily associated with PFM 33, Instability of Spillway Monoliths. Construction activities associated with implementation of RMP 6 could result in a temporary increase to these risks. The biggest driver for these increased risks is the reduction in discharge capacity through the primary spillway as described in Section 6.3.8 and evaluated in Section 6.3.6. The other risks are primarily associated with the potential need for discharge through the half of the unwatered basin for construction. These risks are summarized, along with the plan for mitigation of these risks, in Table 6-4. Although the risks associated with both the left and right side are combined in this table, it should be noted that once the first half of the basin is complete, the likelihood of needing to flow through the other half is reduced significantly. It should also be noted that the construction activities are not listed by order of importance.

#### Table 6-4: Summary of Dam Safety Risk during Construction

<table>
<thead>
<tr>
<th>Construction Activity</th>
<th>Effect on Risk</th>
<th>Mitigation Actions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stilling Basin and Right/Left Side Downstream Cofferdam Construction</td>
<td>Results in flow restriction as described in Table 6-2.</td>
<td>Ensure operations procedures are well documented</td>
</tr>
<tr>
<td>Removal of Existing First Stage Apron Surface, Baffles and End Sill</td>
<td>Loss of energy dissipation for sluice flow, could result in potential scour of passive wedge if more than eight sluices required to pass a flood</td>
<td>Limit number of bays exposed to this condition prior to anchoring of apron</td>
</tr>
<tr>
<td>Anchoring of Existing Apron, Left or Right Side</td>
<td>Open drilled holes present risk to charging foundation if more than eight sluices required to pass a flood</td>
<td>Require contractor to provide packers and grout to fill holes prior to flowing</td>
</tr>
<tr>
<td>Anchor Stilling Weir and Second Stage Basin, Left or Right Side</td>
<td>Open drilled holes present risk to charging foundation if more than eight sluices required to pass a flood, initially only a risk to stability of the weir</td>
<td>Require contractor to provide packers and grout to fill holes prior to flowing</td>
</tr>
<tr>
<td>Rock Excavation for Super Baffle Monolith and Apron Extension, Left or Right Side</td>
<td>Removal of passive wedge reduces sliding resistance of structure</td>
<td>Restrict contractor to excavation only in front of sections with completed existing apron anchors</td>
</tr>
<tr>
<td>Anchoring for Super Baffle Monolith and Apron Extension, Left or Right Side</td>
<td>Open drilled holes present risk to charging foundation if more than eight sluices required to pass a flood</td>
<td>Require contractor to provide packers and grout to fill holes prior to flowing</td>
</tr>
<tr>
<td>Excavation of Random Fill behind Left Stilling Basin Training Wall</td>
<td>Sliding failure of basin walls into excavation or slope failure causes loss of material providing passive resistance to abutment monoliths</td>
<td>Analyze structures for this potential failure</td>
</tr>
<tr>
<td>Tunneling for Toe Gallery (Phase 6)</td>
<td>If cracks form that extend from floor of gallery to foundation, stability of dam could be threatened</td>
<td>Develop a repair method to restore monolithic action</td>
</tr>
</tbody>
</table>
6.4. Uncertainty for RMP 6

6.4.1. Hydraulic Design
Although additional physical hydraulic modeling is proposed to refine the design during the PED phase, this modeling is only expected to better define the loads imparted onto the structures. Phase B hydraulic modeling will also be used to confirm undercut of the second stage apron is not an issue. The layout and configuration of the basin has been confirmed and reviewed by independent hydraulic structures experts.

6.4.2. Geologic (Foundation) Conditions
Extensive investigations and construction at the site in recent years has resulted in relatively well defined geologic profiles. Also, the construction of the auxiliary penstock stilling basin has proven much of the proposed construction methodology for RMP 6 and provided site specific lessons learned. There is one area of potential concern under the second stage basin apron based on notes in the original construction foundation reports that is proposed to be investigated with one or two borings during the PED phase which could result in a localized modification to the design but this modification is likely to only be additional required anchor force due to a weakened plane under the structure.

6.4.3. Structural Design
In addition to the potential changes in the number of required anchors in the dam, there are a few uncertainties associated with the design of the basin and training walls. The structural design for the basin is currently based on a single representative section of one monolith. During the PED phase, each individual monolith will be designed which could result in minor variations across the width of the basin in anchor sizes and spacing but changes in the geometry or scope of the features are not expected. Similarly, but slightly more detailed, the stilling basin training walls are designed based on two cross sections each for the left and right walls, one each upstream and downstream of the baffles. Again, each individual monolith will be designed during PED potentially resulting in minor changes to anchor sizes and layout. The scour protection slabs for the training walls utilized a simplified design based on a conservative estimate of the pressure fluctuations expected from the plunging overtopping flows. There is a potential to optimize this design during PED with a more detailed assessment of the loading.

All of the loads used for the design of the basin features are currently based on theoretical and visual observations of the physical modeling completed to date. Further physical modeling should provide more accurate estimates of the potential loads which will further refine the design. However, given that much of the resisting forces are being provided by anchors due to geometry restrictions of providing more mass with additional concrete, the design refinement will consist of optimization of the anchor quantity, size, and layout. One exception could be the design associated with the second stage basin apron. The current plan includes removal and replacement of the baffles and end sill only. Refined loads could eliminate the requirement for removal of the baffles and end sill, or could result in the need for removal and replacement of part or all of the second stage basin apron slab.

6.4.4. Spoil Quantities and Disposal Sites
As the design details are not yet refined, there is also uncertainty with expected spoil volumes to be generated by the project. Current estimates range from 150,000 cubic yards (CY) to as much as 250,000 CY. This consists of concrete demolition, cofferdam fill, rock excavation for the apron extension, excavation behind left training wall, and other miscellaneous excavation, and includes an appropriate swell factor. Each of these have uncertainties associated with them as follows:

- The concrete demolition quantity is relatively stable with the exception of the second stage basin as described above.
- Optimization of the cofferdam design could reduce the required fill that would need disposed.
Bluestone – Final DSMR

- Rock excavation for the super baffle monoliths and apron extension will vary based on the condition of the bedrock and the final founding elevation of the slab. Based on the relatively well defined geologic profiles, these quantities are not expected to vary significantly from the current design.
- Excavation behind the left training wall could be reduced with a modification to the design for scour protection in this area. If protection is provided at the surface, the excavation and disposal quantities for this feature would be greatly reduced. However, the fill for this area was assumed to come from the rock excavated for the apron extension. Resulting in limited net change to disposal quantities.

Currently the plan is for all disposal will be taken to a commercial landfill because it is a dependable source for disposal. Other more efficient options for material disposal may be available, but the environmental impacts are unknown at this time. Therefore, potential environmental effects of the disposal sites also contribute to uncertainty.

Potential offsite disposal areas were identified based on their currently disturbed nature and some sites identified have been previously utilized for spoil for nearby road construction. A detailed analysis of potential effects would be undertaken during the PED phase at which time, spoil needs would be better understood. Sites which would have no potential for adverse effect would be given priority for use. However, as the sites have not been examined in detail and, depending on quantities, disturbance beyond the existing disturbed footprint may be required, some uncertainty remains with respect to adverse effects to resources.

6.4.5. Mitigation
Mitigation plans have been proposed; however, the details will be refined during PED which may be a source of uncertainty for RMP 6. Through coordination with resource agencies and initial exploration of potential mitigation sites, it is clear that mitigation can be achieved. The final selection for mitigation site(s) and the technical details within these plans have not been developed within the DSMS phase.

6.5. RMP 6 Cost Estimate
The cost estimate for the project has been developed from detail in MCACES Second Generation (MII) cost estimating software. The estimate considers all prior and remaining project costs including engineering, design, and contract supervision & administration. The NWW Cost ATR Certification dated 22 June 2017 presents the Estimated Cost $561.9M at the FY17 price level, the Project First Cost of $897.5M at the FY18 price level, and the Total Project Cost is $1,017.4M fully funded. Within these amounts $323.3M has been spent through FY16, and another $45.15M obligated on existing contracts. See Table 6-5 for a cost summary by contract.
Table 6-5: Total Project Cost

<table>
<thead>
<tr>
<th>Contract</th>
<th>Estimated Cost FY2017 Price Level</th>
<th>Total Project Cost (Fully Funded)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prior</td>
<td>323.3 M</td>
<td>323.3 M</td>
</tr>
<tr>
<td>PRE Prior Expenditures</td>
<td>323.3 M</td>
<td>323.3 M</td>
</tr>
<tr>
<td>Remaining</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dam Safety Modification Study</td>
<td>1.9 M</td>
<td>1.9 M</td>
</tr>
<tr>
<td>Phase 3 Penstock Scour Pad</td>
<td>4.9 M</td>
<td>4.9 M</td>
</tr>
<tr>
<td>Phase 4 278 Multi-Strand Anchors</td>
<td>58.1 M</td>
<td>58.1 M</td>
</tr>
<tr>
<td>Phase 5 Stilling Basin</td>
<td>441.7 M</td>
<td>543.0 M</td>
</tr>
<tr>
<td>Phase 6 Toe Gallery</td>
<td>30.0 M</td>
<td>55.1 M</td>
</tr>
<tr>
<td>Miscellaneous Construction</td>
<td>5.2 M</td>
<td>5.8 M</td>
</tr>
<tr>
<td>Environmental Mitigation</td>
<td>9.6 M</td>
<td>10.7 M</td>
</tr>
<tr>
<td>Recreational Mitigation</td>
<td>5.0 M</td>
<td>5.5 M</td>
</tr>
<tr>
<td>Recreational Park</td>
<td>5.4 M</td>
<td>9.2 M</td>
</tr>
<tr>
<td>Grand Total</td>
<td>885.2 M</td>
<td>1,017.4 M</td>
</tr>
</tbody>
</table>

6.5.1. Construction Cost

A cost estimate was developed for the entire project phases 1 through 6, but this section only focuses on Phases 5 (RMP 6) and Phase 6 (toe gallery). The cost estimate for the recommended plan (RMP 6) was developed using MCACES Second Generation (MII) cost estimating software. The estimate considers all engineering, design, and contract supervision & administration for the DSMS recommendation that is included in this report.
Table 6-6: Cost Breakdown of DSA Activities and Tentatively Selected Plan (RMP 6)

<table>
<thead>
<tr>
<th>Contract / Feature Account</th>
<th>Estimated Cost FY2017 Price Level</th>
<th>Total Project Cost (Fully Funded)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phase 5 Stilling Basin</td>
<td></td>
<td></td>
</tr>
<tr>
<td>04 - DAMS</td>
<td>309.6 M</td>
<td>373.6 M</td>
</tr>
<tr>
<td>30 - PLANNING, ENGINEERING &amp; DESIGN</td>
<td>92.7 M</td>
<td>111.6 M</td>
</tr>
<tr>
<td>31 - CONSTRUCTION MANAGEMENT</td>
<td>39.5 M</td>
<td>57.7 M</td>
</tr>
<tr>
<td>Phase 6 Toe Gallery</td>
<td></td>
<td></td>
</tr>
<tr>
<td>04 - DAMS</td>
<td>18.2 M</td>
<td>27.2 M</td>
</tr>
<tr>
<td>30 - PLANNING, ENGINEERING &amp; DESIGN</td>
<td>5.4 M</td>
<td>12.4 M</td>
</tr>
<tr>
<td>31 - CONSTRUCTION MANAGEMENT</td>
<td>6.4 M</td>
<td>15.6 M</td>
</tr>
<tr>
<td>Miscellaneous Construction</td>
<td></td>
<td></td>
</tr>
<tr>
<td>04 - DAMS</td>
<td>3.1 M</td>
<td>3.4 M</td>
</tr>
<tr>
<td>30 - PLANNING, ENGINEERING &amp; DESIGN</td>
<td>1.8 M</td>
<td>2.1 M</td>
</tr>
<tr>
<td>31 - CONSTRUCTION MANAGEMENT</td>
<td>0.2 M</td>
<td>0.3 M</td>
</tr>
<tr>
<td>Environmental Mitigation</td>
<td></td>
<td></td>
</tr>
<tr>
<td>04 - DAMS</td>
<td>5.1 M</td>
<td>5.5 M</td>
</tr>
<tr>
<td>30 - PLANNING, ENGINEERING &amp; DESIGN</td>
<td>4.1 M</td>
<td>4.8 M</td>
</tr>
<tr>
<td>31 - CONSTRUCTION MANAGEMENT</td>
<td>0.4 M</td>
<td>0.4 M</td>
</tr>
<tr>
<td>Recreational Mitigation</td>
<td></td>
<td></td>
</tr>
<tr>
<td>04 - DAMS</td>
<td>3.7 M</td>
<td>3.9 M</td>
</tr>
<tr>
<td>30 - PLANNING, ENGINEERING &amp; DESIGN</td>
<td>1.1 M</td>
<td>1.2 M</td>
</tr>
<tr>
<td>31 - CONSTRUCTION MANAGEMENT</td>
<td>0.3 M</td>
<td>0.3 M</td>
</tr>
<tr>
<td>Recreational Park</td>
<td></td>
<td></td>
</tr>
<tr>
<td>04 - DAMS</td>
<td>4.0 M</td>
<td>6.0 M</td>
</tr>
<tr>
<td>30 - PLANNING, ENGINEERING &amp; DESIGN</td>
<td>1.1 M</td>
<td>2.4 M</td>
</tr>
<tr>
<td>31 - CONSTRUCTION MANAGEMENT</td>
<td>0.3 M</td>
<td>0.7 M</td>
</tr>
<tr>
<td>Grand Total</td>
<td>497.0 M</td>
<td>629.2 M</td>
</tr>
</tbody>
</table>

The Estimated Cost for the Recommended Plan has been prepared using a 2016 price level. The project First Cost (the cost estimated used to establish the administrative 902 amount) escalates the Estimated Cost to the mid-point of construction. Finally, the fully funded cost estimate or Total Project Cost includes all federal costs: lands, easements, rights of way and relocations; construction features; preconstruction engineering and design; construction management; contingency; and escalation.

The Total Project Schedule is organized by anticipated contract work packages to complete remaining project scope. Contract durations were developed either from real contractual periods of performance or by sequencing the major work activities. There are several key milestones of interest for the Bluestone Dam Safety Project. These are summarized in Table 2 and Figure 1 in the Total Project Cost Estimate...
Baseline (FY17) for the Supplemental Report Evaluation Report (Appendix I). The Dam Safety Risk Reduction Complete milestone defined as the minimal project features required to achieve reduction of risk. The current base project schedule estimates this Dam Safety Risk Reduction milestone competed late FY31 (with 80% confidence completion by early FY34) for Phase 5. The baseline duration for Phase 5 is estimated to be ten years. Site conditions such as care and diversion of water requires non-concurrent sequencing of work which might be otherwise completed concurrently and more efficiently. Further, certain constraints have been assumed due to life risk considerations during construction. These constraints also extend the contract duration. If Phase 6 is determined to be needed the Project Physically Complete is planned for 80% confidence completion by early FY-41.

In order to confirm the least-cost technically acceptable solutions, structural measures will be subject to refinement during the Preconstruction Engineering and Design (PED) phase. Items to be evaluated are categorized by failure mode and include but are not limited to the following:

6.5.2. Non-construction Cost
Non-construction costs typically include Lands and Damages (Real Estate), Planning Engineering & Design (PED) and Construction Management Costs (Supervision & Administration, S&A). PED costs are for the preparation of contract plans and specifications (P&S) are based on staffing plans and historical burn rates. S&A costs are for the supervision and administration of a contract and include Project Management and Contract Administration costs. S&A (as well as Engineering during Construction (EDC)) will be based on staffing plans for the dam safety modification project work. These staffing plans will be based on the empirical implications of work completed to date.

6.5.3. Cost and Schedule Risk Assessment
RMP 6 underwent a constructability evaluation by a team of cost and construction experts. Several suggested changes and additions to the assumptions are now included but the overall conclusion was that the major scope for construction of RMP 6 has been captured. The resulting report from this evaluation is included in Appendix I. In addition to this evaluation a Cost and Schedule Risk Assessment (CSRA) was conducted to capture the uncertainties along with others which could be reasonably be expected during construction. The CSRA is documented within Appendix I.

6.6. Dam Operations after Construction

6.6.1. Permanent Instrumentation Post Construction
Permanent instrumentation will consist of uplift cells and weirs installed to monitor for uplift and seepage. The new divider wall gallery will have one new uplift cell installed in each of its monoliths. There will also be gutters located on each side of the gallery floor. The gutters will collect flows from the spillway basin drain systems on each side of the divider wall and carry the flow to v-notch weirs located near the sump at the downstream end of the divider wall gallery.

Spillway monoliths 26, 29, 39 and 43 will each have an additional three uplift cell cluster installed in the stilling basin. Each group of uplift cells will have one located six feet upstream of the weir, one in the middle of the stilling basin and one located six feet downstream of the baffle monoliths.

Three additional uplift cells will be installed inside each of the right and left training walls, for a total of six training wall uplift cells. Their purpose is to monitor uplift pressures under the training walls and basin for flows through fractures downstream of the main dam (from downstream of the left abutment and between the basins) that cannot be intercepted by a gallery in the main dam.
The piping for all new uplift cells will run between the foundation drain laterals and terminate in the new divider wall gallery. All data from the permanent instrumentation will be manually collected but the instruments will be installed such that they can be easily automated.

Permanent instrumentation for the construction of the toe gallery (Phase 6) has not been determined at this time; however, instrumentation requirements will be developed during PED of Phase 6, if determined necessary.

6.6.2. Post Implementation Operations
After successful implementation of RMP 6, the project will have the ability to release the intended capacity of the spillway, penstocks and sluices. After construction is complete the project will be operated according to an updated water control manual as described in Section 6.3.8.

6.6.3. Revised Penstock Sequencing
Based on 1:65 scale general model observations of flow interactions during Phase A hydraulic modeling, it is suggested that a new operation sequence be implemented for the penstock gate openings after completion of RMP 6. It was observed that for many sequences of openings, a large eddy would form downstream of the right second stage training wall and shed downstream for several hundred feet. This eddy formation and shedding resulted in surging of the right side of the second stage basin. The hydraulic jump would transition in and out of the basin unfavorably. It was determined through trial and error that a sequencing of penstock opening of 1-6-3-5-4-2 eliminated the surging issue (order from left to right looking downstream). It has also been determined that future investigation in to optimizing sequencing to minimize the erosion in the area of highest undermining potential for the second stage basin should be performed.

6.7. Operations, Maintenance, Repair, Replacement, and Rehabilitation (OMRR&R)
OMRR&R costs and considerations for RMP 6 have been contemplated to confirm long-term O&M feasibility. A chart listing pre-construction OMRR&R costs and post-construction OMRR&R costs has been assembled in Table 6-7 for consideration. Annualized costs are shown at FY16 price levels.

The rows in the assembled chart are defined as follows:

- **Baseline costs** include routine maintenance of the dam and recreational facilities, utility costs, contracts for maintenance services, labor funds for project personnel to make flood risk management operations and conduct visitor assistance and water safety patrols, along with District Office Labor funding to provide technical support needed for dam safety, water management, lease agreement management, NEPA and environmental compliance, etc.
- **Periodic Inspection** – costs associated with PI’s and occurring on a 5 year cycle.
- **Periodic Assessment** – costs associated with PA’s and occurring on a 10 year cycle.
- **Anchor Lift-Off Testing** – Approximately 45 anchors will receive lift-off testing. Testing should be first conducted five years after completion, and every ten years thereafter provided no concerns are identified. Lift-off testing will include construction access (platform) to the anchor head location, removing the manhole cover and cap, cleaning the anchorage of wax, and performing lift-off testing of each strand. After lift-off testing the anchor head will be refilled with wax, and the cap and manhole cover replaced in the anchor head recess. The pre-construction costs include lift-off testing of anchors installed in previous phases of work.
- **Instrumentation Monitoring and Maintenance** – Currently, the dam’s instrumentation is monitored on an automated system. These costs assume the project will not fund replacing and/or maintaining the automated system as operation continues into the future. Post-
construction costs assume the monitoring of an additional 33 gallery uplift cells, 80 gallery foundation drains (Phase 6), and four gallery V-Notch weirs.

- **Cleaning of Drains and Uplift Cell Piping** – This work is required every ten years to ensure proper functioning of instrumentation and maintain drain efficiency. This will entail pressure washing of uplift cell piping, and underdrain system piping, and reaming of drains in the toe gallery (Phase 6) and first stage basin. Reaming of the drains in the first stage will require half the basin to be unwatered for access. Post construction costs assume an additional 295 drains in the stilling basin and training walls and 80 in the Phase 6 toe gallery.

### Table 6-7: Annualized O&M Cost (Existing & Post Construction)

<table>
<thead>
<tr>
<th>Annualized O&amp;M Costs</th>
<th>Existing (FWAC)</th>
<th>Post-Construction (RMP 6)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Baseline O&amp;M</td>
<td>$1,500,000</td>
<td>$1,530,000</td>
</tr>
<tr>
<td>Periodic Assessment</td>
<td>$20,000</td>
<td>$20,000</td>
</tr>
<tr>
<td>Periodic Inspection</td>
<td>$20,000</td>
<td>$20,000</td>
</tr>
<tr>
<td>Anchor Lift-off Testing</td>
<td>$200,000</td>
<td>$200,000</td>
</tr>
<tr>
<td>Instrumentation Monitoring and Maintenance</td>
<td>Included in Baseline</td>
<td>$2,000</td>
</tr>
<tr>
<td>Cleaning of Drains and Uplift Cell Piping</td>
<td>Included in Baseline</td>
<td>$16,000</td>
</tr>
<tr>
<td>Total OMRR&amp;R Annualized Costs (10-year cycle)</td>
<td>$1,740,000</td>
<td>$1,788,000</td>
</tr>
</tbody>
</table>

*After conclusion of construction, the downstream recreation area currently used as a staging area for the contractor will be restored to its original condition. Costs to operate and maintain the downstream recreation areas to their original condition prior to DSA construction have been incorporated into Baseline costs for Post-construction, and explains the delta between pre-construction and post construction OMRR&R costs.

Additionally, during construction, the project’s drift and debris tower will be inoperable, as work within the stilling basin will prevent releasing water through the drift and debris tower. Without the use of the drift and debris tower, project personnel must work the drift through the sluice gates, which is much more labor intensive. The associated increase in O&M Costs during construction for drift removal has been estimated at $350,000 per year, resulting in a need for three additional maintenance staff Full-Time Equivalents (FTE’s) and additional funding for contracted services. These cost increases will be incorporated into RMP 6 Cost Estimate.

The preceding table does not capture life cycle cost associated with repair, replacement, and rehabilitation. Additional life cycle costs to consider include those associated with the crest gate PLC, high mast lighting, sump pumps, and maintenance of the former RE office which will be turned over the Operations at the end of construction.

In conclusion, prior to and following construction of RMP 6, OMRR&R Baseline costs are comparable and can be expected to receive funding for completion through the normal O&M Budgetary process. However, the funding to clean the drains and uplift piping ($160,000 every 10 years), along with the anchor lift-off testing ($2M every ten years), must nationally compete for funding amongst other
maintenance needs in USACE, and may or may not receive funding depending on funding levels and risk assessments of these features.

6.8. Applicable Essential USACE Guidelines and Compliance for RMP 6
A list of applicable essential USACE guidelines has been developed for RMP 6 along with a cursory review of existing information. This section provides a brief statement of adequacy for RMP 6 as it complies with essential USACE Guidelines. A number of features designed for RMP 6 do not meet all Essential USACE Guidelines. Table 6-8 shows the waivers that are required to implement RMP 6.

<table>
<thead>
<tr>
<th>Guidelines</th>
<th>Description</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>ER 1110-8-2 (FR)</td>
<td>Dam not designed to pass 100% of IDF</td>
<td>Sections 6.8.1 and 6.8.2</td>
</tr>
<tr>
<td>EM 1110-2-2100</td>
<td>Anchoring of new structures</td>
<td>Section 6.8.11</td>
</tr>
</tbody>
</table>

6.8.1. Hydrologic Design
ER 1110-8-2 requires a high degree of assurance that the dam can pass the IDF without the dam failing. Based on an IDF maximum water surface EL 1,553-ft, the existing top of dam at EL 1,535-ft will be overtopped by up to 18 feet. The dam is not designed for overtopping; this degree of overtopping would likely cause abutment scour leading to breach of the dam. This essential guideline will not be satisfied. Currently, the threshold pool for Bluestone Dam is EL 1,510-ft and once ECRA/FWAC conditions are achieved this threshold pool will be EL 1,520-ft. Upon implementation of RMP 6, this threshold pool will be increased to EL 1,535-ft. However, based on the risk assessment (PFM’s 34 and 35) completed for RMP 6 the risks appear to be sufficiently below tolerable risk guidelines. See Chapter 7 for details of this risk analysis. It should be noted that there is no freeboard considered in the threshold pool since the likelihood of failing the dam with minimum overtopping is so remote.

Engineer Regulation (ER) 1110-8-2 specifies that the selection of the IDF and the design of dam elements necessary to meet minimum safety requirements will conform to Standard 1 as defined in the referenced ER. Standard 1 applies to the design of dams capable of placing human life at risk or causing a catastrophe, should they fail. Standard 1 prescribes that the dam height with appropriate freeboard, spillways, regulating outlets, and structural designs will be such that the dam will safely pass an IDF computed from PMP occurring over the watershed above the dam site.

The IDF for Bluestone Dam is the PMF. However, the TSP is designed to pass a lesser flood equivalent to 70% of the IDF. The TSP is justified because risks are judged to be tolerable. Risk for the TSP is below the tolerable risk guideline for life safety and risks are as low as reasonably practicable. This meets the intent of the IDF policy in that the dam does not pose an unacceptable risk to life safety as defined by the tolerable risk guidelines in Engineer Regulation 1110-2-1156.

Formulating the TSP for a flood less than the IDF was decided on 3-4 March 2016 when the PDT met with the Tier 3 Mega-Project Vertical Governance Senior Executive Board, which included key membership of the DSOG to present the ECRA/FWAC. The meeting concluded that the spillway erosion potential failure mode was the only actionable dam safety issue. Overtopping risks were judged to be tolerable and therefore not actionable at this time. The decision is documented in the Memorandum for Record dated 8 February 2017 (Martin, PE, PMP, 2017). Further coordination with the MSC HH&C CoP representative and the HQ HH&C CoP Leader occurred during an ATR meeting at LRH office on 16 September 2016. Agreement was reached with the HH&C CoP that the proposed waiver was appropriate. The decision is documented in the Memorandum for Record dated 21 September 2016 (Koutsunis, 2016).
The request for waiver to the requirement to pass the IDF is considered part of this DSMS report and decision. Approval of the DSMS report constitutes approval of the policy waiver.

6.8.2. Spillway Capacity

Similarly to the requirements for the hydrologic design, the combined discharge capacity of the primary and auxiliary spillways after implementing RMP 6 is not designed to pass 100% of the IDF; however, the risk assessment (PFM 33) show the risks are sufficiently below tolerable risk guidelines. The key alteration associated with RMP 6 is the modification of the primary spillway and outlet works shared energy dissipator. Because they cannot be easily modified later, the new first stage baffle blocks have been hydraulically designed and verified with physical modeling to function adequately to discharge exceeding the current IDF estimate. No major modifications to the configuration are anticipated for the training walls or second stage basin; these features function acceptably at the threshold flood but have not been evaluated for the IDF.

6.8.3. Outlet Works Capacity

The outlet works, in combination with the primary and auxiliary spillways, do not pass 100% of the IDF. The outlet works (sluices) share the energy dissipator with the primary spillway, although the outlet works contribute less than 10% to the total threshold flow through the stilling basin.

6.8.4. Gate Reliability

Crest gate machinery is aging, known to be failing, and has not been operated under significant (> 10 feet of hydrostatic head) load. The risk assessment performed for the ECRA indicates that gate reliability does increase incremental risk by a half an order of magnitude. A similar increase would be expected for the risks after implementation of RMP 6. This potential increase in the existing and post-implementation risks is being used to actively pursue O&M funding to rehabilitate critical components of the gates and machinery. However, RMP 6 assumes the gates are rehabilitated to the conditions assumed for the FWAC (See Section 3.2.3.2.2). The risk assessment performed for the FWAC indicates that while one or two gates may not operate satisfactorily, overall operation of the project will likely not be impaired.

6.8.5. Water Control

The Water Control Manual (WCM) for Bluestone Dam was last updated in 2005 which is beyond the 5 year requirement to qualify for a DSAC 5 classification. However, a deviation to the WCM defining operations in the interim has been recently approved (2016) and further deviations and updates are planned throughout the dam safety project as described in 3.2.2.2 and in Table 6-2. Additionally, a WCM update will be undertaken as part of implementation of RMP 6.

6.8.6. Water Management Data

The water control data collected for support of operations at Bluestone Lake was evaluated recently. As a result of this evaluation, two stream gages and four precipitation gages were installed above the project.

6.8.7. Sustainability Issues

A site specific PMP and update to the IDF has been completed to supplement this study. The latest sedimentation survey was undertaken in the reservoir in 2007. The results show that the sediment deposition does not appear to be occurring at a rate that is detrimental to project purposes.

6.8.8. Real Estate Interests

All lands required to operate and maintain Bluestone Dam are within Government owned property. Implementation of RMP 6 would not change the required lands, nor does it increase the expected volume or frequency of spillway flow which may create additional hazards to non-Federal interests.
6.8.9. Modeling and Mapping
Modeling, mapping, and consequences have been updated as part of risk assessments during development of the DSMS.

6.8.10. Geologic and Materials Minimum Requirements

6.8.10.1. Geotechnical & Material Design
The dam was reassessed for stability in accordance with EM 1110-2-2100 and EM 1110-1-2908 since changes in the state of practice for stability analysis and selection of rock strength parameters changed significantly from what was standard when the dam was originally designed in the 1930s. The updated stability analysis showed that the dam as originally constructed was unstable for load conditions below the original design pool (EL 1,520-ft), when sliding was evaluated on the open, continuous bedding planes within the foundation. At this time, stability has been improved through the installation of rock anchors and construction of thrust blocks but factors of safety for sliding are still below what is required by essential guidelines for the design pool.

RMP 6 includes additional anchors as required to stabilize the dam. Other features of RMP 6 have been assessed for stability and also requireanchoring to meet sliding factors of safety. Excavation for these features without the use of anchors would be significant creating unacceptable risk during construction. Guidance requires that new structures be designed such that they can be stable without anchors. However, for RMP 6 the construction of the new features are necessary to modify existing dam and stilling basin. Because the new features are for modifications of the existing dam and stilling basin, it is unclear if the proposed anchors for the new features meet the intent of essential guidelines of EM 1110-2-2100. Because of the ambiguity of the intent of the EM 1110-2-2100 a memorandum requesting a waiver for use of these anchors is being routed through LRD to HQUSACE for approval.

To support the stability analysis, investigations including drilling, sampling and laboratory testing programs were performed to provide rock strength parameters and assess fractures and other discontinuities in the foundation. All investigations were performed in accordance with EM 1110-1-1804 Geotechnical Investigations, testing was performed in accordance with Waterways Experiment Station’s (Engineering Research and Development Center’s) Rock Testing Handbook, and rock strength parameters were developed in accordance with EM 1110-1-2908 Rock Foundations.

Rock foundations for new construction including those for the Phase 1 thrust blocks and Phase 3 auxiliary penstock stilling basin were documented by geologic mapping. After the implementation of ER 1110-1-1156, the Phase 3 foundations were also inspected and approved by LRH-DSPC in accordance with the requirements of that regulation to ensure that the design intent was met.

A waiver is being requested for the anchor design depth calculation based on EM 1110-1-2908 dated 30 November 1994. The memorandum serves as the formal request for CECW-CE approval of the alternative methods used to estimate anchor depths for the Bluestone Dam Safety Project. Using Formula 9-3 as proposed in Chapter 9 of EM 1110-1-2908 for design of anchors for the Bluestone Dam Safety Project produces embedment depths with constructability issues. As such, the memorandum dated 30 March 2017 request to use Formula 9-4 with the total of all anchor loads substituting for the single anchor load to calculate the anchor depth and then using 3-D MicroStation to confirm the design or 3-D MicroStation alone to determine anchor embedment depths. Both alternative methods rely on the weight of the rock mass to resist anchor forces as does Formula 9-3. (Morgan, P.E. & Martin, P.E., PMP, 2017)
6.8.10.2. Reservoir Rim

It is unknown if the reservoir rim has been assessed or evaluated for stability. Additional research will be required to determine what, if any, assessments have been done. If any assessments exist, an update may be required to reflect changes to the reservoir such as evaluation of new man-made structures or fills and consideration of valley walls above the original design pool that would be inundated by the updated IDF.

6.8.10.3. Conduits

There are no conduits at this project.

6.8.10.4. Drainage Features

The existing drainage features for Bluestone Dam are a drainage gallery in the main dam body and underdrain system under the spillway apron and the second stage weir apron. While the existing gallery drains have been reamed as recently as 2009 the gallery system has been significantly compromised with over 10% of drains lost to interception and grouting related to anchor installation during Phase 2B and Phase 4 (approximately 50% complete). It is anticipated that drains will continue to be lost throughout the remainder of Phase 4 construction. The spillway apron drainage outlet pipes have been observed to be filled with silt. Because the two apron drainage systems are not constructed in such a way that they are maintainable they are assumed to be ineffective. The existing system is compromised such that it likely no longer meets essential guidelines.

To remediate the compromised drainage system an underdrain system within the stilling basin will be constructed, along with a second (toe) gallery within the toe of the spillway monoliths, if needed. All new drainage system components can be inspected and validated for proper function and serviced as needed to ensure efficiency over the service life of the structure. This system meets the requirements of EM 1110-2-2200 Gravity Dam Design and EM 1110-2-1901 Seepage Analysis and Control for Dams. All components (piping, sumps and pumps) will be sized to function adequately for the PMF since the incremental cost difference, if any, is likely to be insignificant.

6.8.10.5. Filters

The project has no filters.

6.8.10.6. Earthquake Resistance

The dam is situated in an area of moderate tectonic activity. Seismic parameters were selected from a 1983 site specific study and a 1996 regional study. The site specific study was conducted in accordance with a previous version of ER 1110-2-1806 Earthquake Design and Evaluation for Civil Works Project. The regional study was developed after the 1995 revision to that guidance. Seismic design parameters were taken from both the 1983 and 1996 seismic studies by selecting the more conservative results of peak ground accelerations (PGA) for the maximum credible earthquake (MCE) and operating basis earthquake (OBE) from each study. The PGA for the MCE was selected as 0.32g, taken from the 1996 seismic study. The PGA for the OBE was selected as 0.09g, taken from the 1983 seismic study. Although the newer 1996 study provided a reduced OBE PGA of 0.01g, the 1983 study value of 0.09g has already been used in previous studies to evaluate the dam and, for this reason, it has been selected to remain the design value. All selected design values meet essential guidelines requirements.

Seismic load conditions for the dam have been assessed in accordance with EM 1110-2-2100 and EM 1110-2-6053; seismic loads were shown to not control the design. In 2013, a study was completed where six representative monoliths of the dam were evaluated for internal stresses and external stability when loaded with the MCE and OBE. The subject representative monolith sections included all existing and proposed stabilizing features (anchors, toe blocks, etc.) of the post Phase 4 project, except for the spillway.
For the spillway, the section evaluated only included the spillway anchors, the existing spillway apron with anchors, and a rock wedge downstream of the apron. The specific work included development of two-dimensional finite element models for the six representative monolith sections. The developed models were used to perform two-dimensional seismic analyses of the representative monolith sections loaded with the OBE and MCE ground motions. Conclusions for the study using the modeling results indicated that the entire dam with the post Phase 4 stabilizing elements is adequate with respect to internal stresses and internal and external stability when loaded with the MCE and OBE. Seismic evaluation for features associated with the stilling basin will be conducted in PED. It is anticipated that this evaluation will not result in changes in design for these features.

6.8.11. Structural Minimum Requirements
EM 1110-2-2100, Stability Analysis of Concrete Structures, 1 December 2005 requires approval from CECW-E to use new anchors to provide sliding stability to new concrete structures. As this study addresses modification of the existing dam (considered an existing structure), it is unclear whether a waiver request is required for anchoring of the new features of the recommended plan. Given the ambiguity in the intent of the guidance regarding what constitutes a “new” structure, a waiver request has been submitted for installation of the anchors for new concrete features that are part of the recommended plan (RMP 6). The new concrete features to receive new active (tensioned) anchors are the super-cavitating baffles, apron extension, and divider wall. This waiver request has been developed for the installation of the new anchors as outlined in Paragraphs 8-1 and 8-7b of EM 1110-2-2100. A memorandum requesting waiver for use of these anchors is being routed through LRD to HQUSACE for approval. See Table 6-9 and associated Figure 6-16 for features of RMP 6, anchor type required, tensioning requirements and waiver requirements.

Table 6-9: Required Anchor Types and Tensioning & Waiver Requirements for RMP 6 Features

<table>
<thead>
<tr>
<th>Feature</th>
<th>New/ Existing</th>
<th>Anchors</th>
<th>Active/ Passive</th>
<th>Waiver Required</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spillway/Dam</td>
<td>Existing</td>
<td>Strand</td>
<td>Active</td>
<td>No</td>
</tr>
<tr>
<td>Spillway Apron</td>
<td>Existing</td>
<td>Bar (Gr 150)</td>
<td>Active</td>
<td>No</td>
</tr>
<tr>
<td>Super Cavitating Baffle</td>
<td>New</td>
<td>Strand</td>
<td>Active</td>
<td>Yes</td>
</tr>
<tr>
<td>Upstream Apron Extension</td>
<td>New</td>
<td>Bar (Gr 150)</td>
<td>Active</td>
<td>Yes</td>
</tr>
<tr>
<td>Downstream Apron Extension</td>
<td>New</td>
<td>Bar (Gr 150)</td>
<td>Active</td>
<td>Yes</td>
</tr>
<tr>
<td>Stilling Weir</td>
<td>Existing</td>
<td>Strand</td>
<td>Active</td>
<td>No</td>
</tr>
<tr>
<td>Stilling Weir Apron</td>
<td>Existing</td>
<td>Bar (Gr 150)</td>
<td>Active</td>
<td>No</td>
</tr>
<tr>
<td>Training Walls</td>
<td>Existing</td>
<td>Strand</td>
<td>Active</td>
<td>No</td>
</tr>
<tr>
<td>Divider Wall</td>
<td>New</td>
<td>Strand</td>
<td>Active</td>
<td>Yes</td>
</tr>
<tr>
<td>Training Wall Overtopping Protection</td>
<td>New</td>
<td>Bar (Gr 60)</td>
<td>Passive</td>
<td>No</td>
</tr>
</tbody>
</table>
Also as noted in EM 1110-2-2100, the recommended plan is to be designed to the maximum design flood (MDF). However, for this study the MDF for the recommended plan is a lesser flood equivalent to 70% of the IDF as noted in the H&H policy waiver request. Therefore, an additional waiver is not needed for the structural design based on the 70% IDF.

6.8.11.1. Structural Data
Historical structural design documents, as-built drawings, material specifications, foundation reports, and construction reports are available at the project and district office. All available construction data on the dam is currently being integrated into a data management system which also details the geometry of the structure.

6.8.11.2. Structural Condition Assessment
Periodic inspections (PI) have been conducted on Bluestone Dam as required every five years with the most recent in 2014. The stilling basin was last unwatered during the 2004 PI. During the 2009 and 2014 inspections, the basin could not be unwatered due to a dewatered construction area behind the right training wall preventing drainage of the stilling basin through the training wall sluice. Service bridges and hydraulic steel structures are inspected under separate programs on a regular schedule and the findings included in the PI reports.

Because of excessive expansion of service bridge girders, an investigation was conducted in 2002 to test for alkali-silica reactive (ASR) aggregate in concrete from various locations of the dam. While the investigation did indicate the presence of reactive aggregate, the conclusion was that the level of silica in the aggregate was low and expansion had most likely stopped. The load rating for the service bridges, which seem to be most affected by ASR, has not been affected by the expansion.

Many of the post-tension anchors installed in the dam as part of the DSA program work are due for 10 year liftoff testing. This testing has not been completed. However, funding for portions of liftoff testing is available and additional funding has been requested for FY17 to complete the required testing.

6.8.11.3. Structural Stability Evaluation
As a concrete gravity structure, failure of any monolith could precipitate failure of the project. Work has been occurring under the DSA program to improve the stability of the structure when analyzed in
accordance with EM 1110-2-2100. At the conclusion of Phase 4 construction, the dam has been shown to be stable to an Interim Maximum Flood Control Pool (IMFCP) of EL 1,521-ft, correlating with the top of the crest gates. Because of uncertainty regarding the stability of the dam apron without anchors, crest flow was not considered in the Phase 4 design. Computations were completed to design Post Phase 4 anchors for the dam and apron (assuming no rock scour of the stilling basin) which would bring the project to its authorized pool of EL 1,542.2-ft; these computations may be found in the Phase 4 DDR (US Army Corps of Engineers, Bluestone Dam Phase 4 Design Documentation Report (DDR), 2012). Ultimately, the post Phase 4 design for the stilling basin will be largely superseded by the design of RMP 6. The designs for the dam monoliths will be reevaluated to ensure they are adequate given the new basin construction and other changes in design assumptions/parameters provided in Section 6.2.10. As described in Section 6.8.10.6, seismic considerations were included in previous design modifications and future design modifications.

6.8.11.4. Structural Strength and Serviceability Evaluation

The crest gates were last inspected in 2003 per requirements of ER 1110-2-8157; given the environmental conditions and frequency of loading, inspections are on a 25 year cycle. Gates were analyzed for the BCRA. The analysis was conducted using applicable guidance at the time: EM 1110-2-2701 and EM 1110-2-2105. The sheave pin plate was found to be overstressed in extreme conditions which could result in failure of the gate to open. However, the analysis found the gate should not fail catastrophically resulting in uncontrolled release.

Sluice gates are typically inspected during periodic inspections, but are not included in the hydraulic steel structures inspection program.

Penstock gates were designed in accordance with guidance in effect during the design phase of DSA Phase 1. These gates have not been inspected, but are planning to be included in future inspections in accordance with ER 1110-2-8157. Given their low frequency of loading, inspections will likely be on a 25 year cycle.

The dam piers have been evaluated and found to be structurally adequate.

Service bridges are inspected on a five year interval in accordance with ER 1110-2-111; no structural deficiencies have been noted.

6.8.12. Instrumentation

Potential failure modes for the spillway monoliths, training walls, divider wall, cofferdam cells and granular cofferdam were identified. Temporary and permanent instrumentation was designed around the characteristics of the structures as well as these identified potential failure modes; uplift, embankment seepage, sliding, deformation and rotation. Automated data collection and a reading frequency of one data set every fifteen minutes were selected to address the high level of risk associated with the cofferdam during construction. Redundant instruments as well as a backup communications link were incorporated into the design to ensure critical data acquisition and transfer were available at all times.

Temporary and permanent instrumentation additions, coupled with existing instrumentation enable the appropriate level of monitoring and evaluation of the dam both during the construction period and post construction under all operating conditions.

A surveillance and monitoring plan will be implemented during construction to ensure proper visual observations occur at specified intervals. Check sheets will be developed to insure visual inspections encompass the appropriate details associated with each project feature. The existing Instrumentation Observation Schedule will be revised to incorporate the permanent instrumentation additions.
Applicable current standards applied include ER 1110-2-1156 Engineering and Design Safety of Dams, EM 1110-2-4300 Instrumentation for Concrete Structures, EM 1110-2-1908 Instrumentation of Embankment Dams and Levees, and ER 1110-2-8152 Planning and Design of Temporary Cofferdams and Braced Excavations.

### 6.8.13. Operations and Maintenance

Bluestone Dam is currently rated as yellow in the Dam Safety Program Management Tool (DSPMT) scorecard, as described in ER 1130-2-530, with an overall score of 78 out of 100. The components of the total score are summarized in Table 6-10. Implementation of RMP 6 is not anticipated to significantly alter this score.

#### Table 6-10: Bluestone Dam DSPMT Scorecard Summary

<table>
<thead>
<tr>
<th>Category</th>
<th>BLN Score</th>
<th>Max Score</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inspections and Evaluations</td>
<td>16</td>
<td>30</td>
<td>PI timeliness and essential USACE guidelines assessment</td>
</tr>
<tr>
<td>Project Instrumentation</td>
<td>18</td>
<td>18</td>
<td></td>
</tr>
<tr>
<td>Project Response Preparedness</td>
<td>10</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>Agency and Public Response Preparedness</td>
<td>15</td>
<td>15</td>
<td>Identified IRRM’s not complete</td>
</tr>
<tr>
<td>IRRM’s</td>
<td>19</td>
<td>25</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>78</td>
<td>100</td>
<td></td>
</tr>
</tbody>
</table>

### 6.9. Environmental Impacts and Considerations

#### 6.9.1. Summary of Environmental Impacts from Implementation of RMP 6

The construction of a non-permanent cofferdam for stilling basin dewatering would have direct and indirect adverse impacts on downstream botanical, wildlife, water, and aquatic resources through clearing of riparian vegetation, disturbance of aquatic habitat, downstream flow alteration (including changes in water depth and velocity) and increased suspended solids and sedimentation. The use of only eight of the sixteen sluice gates to pass water through the dam would result in upstream adverse impacts to botanical, water, and aquatic resources, as well as recreation resources, by causing an increase in the frequency, duration, and elevation of out of pool conditions within Bluestone Lake. This change in lake inundation would result in insignificant increased sedimentation and resulting vegetation stress, and would cause more frequent closure of recreational sites such as campgrounds and trails. All of these impacts would be long-term given the 8 to 10-year construction duration of the TSP; however, most would be non-permanent. With the exception of the significant direct and indirect impacts to aquatic resources downstream of the dam due to construction and dewatering of the cofferdam, removal of the public fishing pier downstream, and moderate disruption of upstream recreation due to inundation, the other upstream and downstream impacts range from negligible to moderate.

Construction of the TSP features would cause moderate adverse impacts on the ambient noise environment and local air quality, causing long-term but non-permanent adverse impacts to quality of life for nearby residents and recreational facilities. Long-term but non-permanent impacts are defined as those that would last more than one year but cease within one year of construction completion (estimated between 8 to 10 years). Construction noise would also continue to impact the use of the area’s terrestrial
habitat by some terrestrial wildlife species. Visual impacts to the dam and tailwater area would be minimal to moderate, and long-term. Cultural resources upstream of the dam would be at an increased risk of inundation during construction, while no adverse impacts to cultural resources are anticipated downstream of dam.

6.9.2. Summary of Environmental Commitments & Mitigation Measures for RMP 6
This section of the report highlights the major commitments that are required to construct RMP 6. The details and descriptions of all the commitments and measures can be found in the SFEIS (Appendix K). Chapter 7 of the SFEIS outlines these measures.

- Seasonal restrictions for tree clearing are required to prevent impacts to bats and birds (clearing may only occur between November 15th through March 31st)
- Truck traffic passing through residential areas would be limited to the hours of 9:00 a.m. through 2 p.m. Monday through Friday during construction.
- Proper and routine maintenance of all vehicles and other construction equipment would be implemented to ensure that emissions are within the design standards of all construction equipment. Dust suppression methods should be implemented to minimize fugitive dust.
- USACE would be committed to active and prolonged outreach to media outlets to inform the public of when recreational facilities would be unavailable as a measure to mitigate for impacts.
- In order to minimize the risk of introduction of invasive mussels into the New River, all construction boats would be decontaminated prior to use within the New River.
- USACE is designing a bypass system that would transmit water from the left-side (west-side) of the stilling basin to the non-operational side of the stilling basin to evenly disperse water downstream of the non-operational side, which would lessen the impact of flow alteration during the lowest flow allowed through the dam (610 cfs).
- Stipulations detailed in the memorandum of agreement (MOA) with the State of West Virginia State Historic Preservation Office and the Advisory Council for Historic Preservation will be implemented for cultural resources/historic properties.
- Construction of a new fishing pier downstream of the dam on either the left or right descending banks.
- Replacement or protection of the existing public fishing platform/pier on the left descending bank.
- Replacing an ADA fishing pier just upstream of the dam.
- Vegetation Monitoring
- Implementation of various Best Management Practices (BMPs)
- Aquatic mitigation measures can be found in Section 6.2.12.1.
- Vegetation mitigation measures can be found in Section 6.2.12.3.
- Recreation mitigation measures can be found in Section 6.2.12.2.

6.10. Real Estate Considerations
Additional land required for the project is approximately 2.96 acres of temporary work area easement across four landowners. This land is needed for contractor laydown and staging. It is estimated that the temporary work area easement is required for 10 years in order to facilitate the ongoing construction at the dam. Other real estate may need to be acquired for a spoil/disposal area to be used during construction, but it has yet to be determined where that area will be located. Real estate may also have to be acquired for mitigation purposes. Once lands required for spoil and mitigation have been identified, a subsequent supplement to the Real Estate Design Memorandum will be prepared and forwarded for
approval. Please refer to Supplement #4 to the Real Estate Design Memorandum in Appendix L for additional real estate information.

6.11. Cost Sharing Consideration
The original project was built at full Federal expense. Therefore, there are no non-Federal cost sharing requirements associated with implementation of RMP 6. The Federal government is fully responsible for the modifications, operation, and maintenance of Bluestone Dam.

While RMP 6 is the only reasonable and cost effective alternative identified to address the goals and objectives of the DSMR in terms of incremental risks, several opportunities exist for additional studies and/or actions to be taken to reduce non-breach risks with or without further action to reduce incremental risks.

Hydrologic investigations conducted during the study also revealed potential opportunity for non-breach risk reduction through the development of an optimized induce surcharge schedule for major floods that incorporates the additional safe storage volume and peak discharge attenuation.

For example, the original IDF was less than the design discharge for RMP 6. In addition, the design maximum headwater was EL 1,523-ft. The current operations in the WCM includes an induced surcharge curve for flood operations. This operation schedule was developed under the context of the maximum flood would have a peak discharge of 430,000 cfs and a peak pool of EL 1,523-ft; through implementation of RMP 6, the maximum safe discharge has been increased to 860,000 cfs and maximum pool elevation of EL 1,536-ft. This allows for additional flood risk benefit by utilizing additional surcharge storage from elevation EL 1,523-ft to EL 1,535-ft, although the reasonable maximum surcharge pool elevation needs to consider uncertainty in predictions and associated freeboard. This is a relative increase of 15% of total project storage utilized for flood risk benefit.

Increasing the surcharge would result in a higher pool and a slight increase in incremental risk. Additional analysis would be necessary to fully understand the tradeoffs in risks this modification would have. Various alternative surcharge curves could be explored to find and optimize risk tradeoffs to ensure residual risks are not increased. A modification to the existing WCM would likely be required for the change. This modification to surcharge curve would require a modification to the WCM and may also require additional authorization to implement.

Though non-structural measures and risk preparedness efforts by the downstream population is forecasted and expected as described in the FWAC in Section 3.2.3.8, these activities could potentially be enhanced and/or accelerated through existing authorities granted USACE, if supported and desired by local interests. The Planning Assistance to States (PAS) or Flood Plain Management Services (FPMS) could provide assistance to help downstream communities consider strategies to reduce flood hazards through resiliency planning efforts and or floodplain management planning to include but not limited to development and installation of Flood Warning and Emergency Evacuation Plans (FWEEPs).

It is possible that one or more small flood risk management projects (both structural and/or non-structural) could be studied and implemented to reduce flood hazards through the USACE Continuing Authorities Program (CAP) Section 205 of the Flood Control Action of 1948. Section 205 studies are funded 100% up to $100,000. Any study costs above that are cost-shared 50% with a non-Federal sponsor. The sponsor must also contribute 35% of the total project design and construction cost as cash, in-kind services or lands.
It is also possible that a more comprehensive flood risk management project could be identified through a newly authorized general investigations feasibility study. Typically, these studies require 3 years and approximately $3M to develop and are cost-shared with a non-Federal sponsor, as is the implementation of any measures recommended within the Federal interest.

Finally, it is possible that other agencies and/or programs outside of the scope of USACE could be implemented to reduce non-breach risks. Hazard mitigation programs or similar effort may be undertaken by local emergency management officials with or without the assistance of programs managed by agencies such as the Federal Emergency Management Agency (FEMA). Local municipalities may also take actions such as upgrading evacuation and response planning beyond those efforts assumed for the FWAC.
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7. SELECTED PLAN RISK ASSESSMENT

7.1. Incremental Risk

The risk that can be attributed to the presence of the dam and caused by breach of the dam, component malfunction or misoperation, is referred to as the incremental risk. To guide the process of examining and judging the significance of estimated incremental risks, it is compared against and discussed in context with USACE tolerable risk guidelines for life safety. While meeting or achieving the tolerable risk guidelines is the goal for all risk reduction measures, the outcomes of a risk assessment are considered to be inputs, along with other considerations, to the risk management decision process.

RMP 6 was chosen prior to conducting the With-Project quantitative risk assessments on a final array of RMPs. The results presented in this chapter are only for the RMP 6. A quantitative risk assessment was not completed for any of the other RMPs; however, they were evaluated qualitatively.

7.2. Annualized Probability of Failure

The annualized probability of failure (APF) is estimated for those failure modes associated with incremental risk associated with all loading or initiating event types. A total APF less than 0.0001 per year is considered tolerable provided the other tolerable guidelines are met. The basis to take action to reduce or to better define the risk diminishes as the APF estimate becomes smaller than 0.0001 per year. A total APF greater than 0.0001 per year is considered unacceptable except in exceptional circumstances. The basis to take action to reduce or to better define the risk increases as the estimate becomes greater than 0.0001 per year. As shown in Table 7-1, the calculated APF for RMP 6 is less than the tolerable limit.

Table 7-1: RMP 6 Risk Assessment Results

<table>
<thead>
<tr>
<th>Risk Driver Potential Failure Mode</th>
<th>APF</th>
<th>N</th>
<th>AALL</th>
<th>Contribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>PFM 33 – Spillway Monolith Instability</td>
<td>2.92E-08</td>
<td>587</td>
<td>1.71E-05</td>
<td>3.38%</td>
</tr>
<tr>
<td>PFM 34 – Non-Overflow Monolith Instability</td>
<td>6.50E-08</td>
<td>588</td>
<td>3.82E-05</td>
<td>7.55%</td>
</tr>
<tr>
<td>PFM 35 – Abutment Monolith Instability</td>
<td>7.58E-07</td>
<td>595</td>
<td>4.51E-04</td>
<td>89.07%</td>
</tr>
<tr>
<td>Total</td>
<td>8.52E-07</td>
<td>594</td>
<td>5.06E-04</td>
<td>100.00%</td>
</tr>
</tbody>
</table>

7.3. Incremental Life Safety Risk

7.3.1. Average Annual Life Loss

The average annual life loss (AALL) associated with the incremental risk is evaluated based on the limit value of 0.001 for estimated annual loss of life per year. The value of this metric for a dam is estimated from all failure modes associated with all loading or initiating event types and considering all exposure conditions associated with life loss. A total AALL less than 0.001 lives per year is considered tolerable provided the other tolerable guidelines are met. The basis to take action to reduce or to better define the risk diminishes as the estimate becomes smaller than 0.001 per year. A total AALL greater than 0.001 lives per year is considered unacceptable except in exceptional circumstances. The basis to take action to reduce or to better define the risk increases as the estimate becomes greater than 0.001 per year. As shown in Table 7.1, the AALL for RMP 6 is below this guideline. Table 7.1 also includes the weighted average for incremental life loss $\bar{N}$ which is calculated by dividing the AALL by the APF. The f-$\bar{N}$ chart shown in Figure 7.1 illustrates how the estimates for the failure modes driving the risk and the incremental project risk versus the AALL guideline for the analyzed conditions and scenarios. Again, it can be seen that by implementing RMP 6, the risk associated with the spillway, abutments and non-overflow monoliths will be below tolerable risk guidelines. Even if the conservative assumption was made that the new drainage gallery at the toe of the dam was completely ineffective and the probability of failure for PFM
increased an order of magnitude, the expected value of the total risk for Bluestone by implementing RMP 6 would still be below tolerable risk guidelines. Further discussion on sensitivity analysis done on drain efficiency of the new drainage gallery can be found in Section 2.5.1 of the TSP Risk Assessment Write up in Appendix A.

Figure 7-1: \( f-\bar{N} \) Chart displaying APF and AALL for Incremental Risk
The dashed line boxes shown around the mostly likely values in Figure 7.1 indicate the sensitivity of the analysis to the hydrologic loading and the consequences and a degree of uncertainty with the risk assessment. These boxes should not be confused with defining all uncertainty. The true uncertainty for RMP 6 was not totally quantified. Per paragraph 1.11.12 of ER 1110-2-1156, the level of effort and scope of risk assessments will be scaled to provide an appropriate level of confidence considering the purpose of the risk management decision. There is uncertainty associated with the hydrologic loading and the consequences as well as the structural analysis. As noted in Section 2.5.3 of the with-project assessment in Appendix A, there are numerous conservative assumptions in the analysis of the spillway monoliths and new stilling basin. For instance, a stilling basin slab is assumed to be removed once the calculated floatation factor of safety is less than one and the foundation is assumed to erode down to the sliding plane and all the way back to the monolith. Three dimensional effects in relation to localized failures within the stilling basin and movement of the monoliths are ignored in the assessment of the spillway monoliths as well. There is also uncertainty related to the assessment of the abutment monoliths, as the bottom of the scour hole was conservatively assumed to be a horizontal top of rock for the passive wedge. The probability of breach during the IDF loading was also assumed to be 0.9 despite the fact that the scour hole created during such event is not expected to be any larger than loads which are considered to have a 0.1 probability of breach. Most of these conservative assumptions are only relevant to the TSP. Refinement of any or all of these conservative assumptions would inject even more uncertainty into the assessment. Quantifying this uncertainty, however, would only reduce both the most likely value and upper bounds of the boxes shown in Figure 7.1. Given that the most likely values are already below tolerable risk limits, further refinement of the uncertainty would not affect any decisions made based on the risk assessment. Per ER 1110-2-1156 the team feels that the appropriate level of confidence has been provided to make risk management decisions without refining any of the conservative assumptions made.

### 7.3.2. Probability Distribution of Potential Incremental Life Loss
This societal incremental risk guideline is represented by a probability distribution of the estimated annual probability of potential life loss from dam failure or breach, for all loading types and conditions and all failure modes and all population exposure scenarios. This is displayed as an F-N chart which is a plot of the annual probability of exceedance (greater than or equal to) of potential life loss (F) versus incremental potential loss of life (N) associated with the incremental flood risk. The F-N chart displays the estimated probability distribution of life loss for a reservoir encompassing all failure modes and all population exposure scenarios for a particular dam for the incremental inundation risk. Dams with failure risks that plot above the tolerable risk limit on an F-N chart, as represented by the dashed line, are considered to have an unacceptable level of risk. As shown in Figure 7.2, the probability distribution of incremental life loss for RMP 6 is below the societal risk guideline.

### 7.3.3. Individual Incremental Life Loss
The individual risk (IR) is represented by the probability of life loss for the identifiable person or group by location that is most at risk. IR is evaluated against USACE individual life safety risk guideline applied to all failure modes associated with all loading or initiating events, with due regard for non-mutually exclusive failure modes, and non-zero population at risk (PAR). The individual risk to the identifiable person or group by location, that is most at risk, should be less than a limit value of 0.0001 per year, shown as a dashed line on the IR chart, except in exceptional circumstances. It is important to note that it will always be true that APF ≥ IR and that the USACE tolerable risk guideline for both APF and IR is 0.0001 per year. For Bluestone Dam, the individual or group most at risk is the permanent PAR that resides in Bellepoint, West Virginia. The same exposure rate (100%) and fatality rate (91%) which was used in the ECRA was used to calculate the IR for RMP 6. From Figure 7.2 it can be seen that the individual tolerable risk limit is satisfied by RMP 6.
7.4. Incremental Economic Risk

The incremental economic risk is summarized for RMP 6 in Table 7-2.

Table 7-2: Incremental Property Damage

<table>
<thead>
<tr>
<th>Risk Driver Potential Failure Mode</th>
<th>Average Incremental Property Damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>PFM 33 – Spillway Monolith Instability</td>
<td>$2,588,418,176</td>
</tr>
<tr>
<td>PFM 34 – Non-Overflow Monolith Instability</td>
<td>$2,589,003,554</td>
</tr>
<tr>
<td>PFM 35 – Abutment Monolith Instability</td>
<td>$2,609,566,196</td>
</tr>
<tr>
<td>Total</td>
<td>$2,607,272,616</td>
</tr>
</tbody>
</table>
7.5. Non-Breach Risk
The risk in the reservoir area and affected downstream floodplain due to normal operation of the dam or overtopping without breach is referred to as the non-breach risk. The average annual non-breach life safety flood risk is 1.17E-03 lives per year and the average annual non-breach economic flood risk is $895,000 per year. A probability distribution of the annual probability of non-breach life loss is provided in Figure 7.3. The dashed lines in Figure 7.3 do not have the same meaning as the societal tolerable risk limit but rather serve as a reference line for communicating the non-breach life safety risk and a means of comparing to the incremental life safety risk.

![Figure 7-3 – F-N Portrayal of Non-Breach Life Safety Risk](image-url)
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8. **Reviews**

**8.1. USACE Value Engineering Study**

The Value Engineering (VE) Workshop was performed 18 – 21 November, 2013 at LRH. The USACE six-step Value Engineering Job Plan was used to facilitate and document the workshop. The objective of this workshop was to incorporate VE analysis into the development of alternative measures for proposed structural modifications and possible non-structural actions to reduce the level of dam safety risk for Bluestone Dam. At the time of the VE study, the project was early in the Dam Safety Modification Study. This effort resulted in recommendations that may improve project performance, avoid initial or future costs and expedite project implementation.

The VE Team was comprised of subject matter experts from a number of districts across the country as well as LRH project delivery team and operations staff. As part of the workshop, the Team identified important project issues and noted pertinent field observations. A Function analysis (F.A.S.T.) diagram was developed as part of the workshop. ‘Brainstormed’ project improvement ideas were compiled and screened. A lists of all the ideas (Speculation List) categorized by their disposition (developed or not developed) can be found in the Value Engineering Report Appendix P Reviews. The VE Team also provided recommendations from their analysis. The recommendations and study team responses are contained at the end of the report located in Appendix P. The most significant recommendations and responses are summarized in Table 8-1. Please note the study team responses are updated to reflect considerations throughout the study process.

<table>
<thead>
<tr>
<th>VE Team Recommendation</th>
<th>Project Study Team Response</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Recommendation 1: Optimize configuration and features of proposed stilling basin modifications</strong></td>
<td>This has been done throughout the alternative formulation process. Standard engineering process.</td>
</tr>
<tr>
<td><strong>Recommendation 2: Complete parapet wall</strong></td>
<td>This recommendation was previously proposed as part of the DSA project and was reconsidered in the formulation of the DSMR as an ALARP measure within RMP 6.</td>
</tr>
<tr>
<td><strong>Recommendation 3: Install operable gates on penstock intakes</strong></td>
<td>Installation of operable gates on the penstocks was considered as a potential opportunity but was eliminated for care and diversion of water.</td>
</tr>
<tr>
<td><strong>Recommendation 15: Create hybrid alternative by deepening either upstream or downstream stilling basin to achieve energy dissipation</strong></td>
<td>The deep basin at the toe of the dam created too much construction risk. RMP 4 moved the deep basin downstream and was found not to be as efficient of RMP 6.</td>
</tr>
</tbody>
</table>

**8.2. Constructability Evaluation (CE) Review**

The CE Team, under the leadership of Mr. Matthew Sheskier, P.E., was formed to provide a review of the design and draft cost estimate for RMP 6 in accordance with ER 1110-2-1156. Chapter 22 of the ER provides guidance on the nature and scope of the CE review. A briefing on the project history, condition, and the selected risk management alternative was held at the Bluestone Resident Engineers Office on 14...
June 2016. Follow-on discussions were held at LRH Offices on 15 and 16 June 2016. An out-brief at LRH was conducted by the CET on 16 June 2016. The PDT briefed RMP 6 that has been developed to address actionable failure modes identified by the Risk Cadre, PFM 34/35 Overtopping Scour, and PFM 33 Spillway Scour. This Review was requested to evaluate the constructability and other aspects with regard to the implementation of the selected risk management measures. The CE team concluded that the TSP as presented for CE is constructible and feasible. The overall findings are presented in the Constructability Evaluation Report found in Appendix P.

8.3. District Quality Control
In accordance with EC 1165-2-214 as updated by ECB 2016-4, a District Quality Control (DQC) Review was completed and details can be found in Appendix P along with the DQC certification. Each reviewed appendix had its own certification sheet and can be found in the front of those appendices.

8.4. Agency Technical Review
During the ATR, compliance with established policy principles and procedures, utilizing justified and valid assumptions, were verified by the Team. This included review of: assumptions, methods, procedures, and material used in analyses, alternatives evaluated, the appropriateness of data used and level obtained, and reasonableness of the results, including whether the product meets the customer’s needs consistent with law and existing USACE policy. Judging from the ATR Team’s review of the documents, the DQC appears to have accomplished the intended purpose. Comments resulting from the ATR were broken down into subject areas so the PDT and ATR team members could focus on resolution of issues. Table 8-2 presents the different subject areas, the number of comments in each area, and a brief capsule describing the major issues and any significant decisions that helped move toward resolution of the comments. The actual ATR comments are included in Appendix P.
### Table 8-2: Summary of ATR Issues and Resolution

<table>
<thead>
<tr>
<th>Subject</th>
<th>No. of Comments</th>
<th>Description of Major Issues and Significant Decisions moving toward Resolution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cost</td>
<td>3</td>
<td>ATR team should review CSRA risk register.</td>
</tr>
<tr>
<td>Model Testing</td>
<td>4</td>
<td>The model testing completed to date supports the DSMS level design for the recommended plan.</td>
</tr>
<tr>
<td>Design Level</td>
<td>8</td>
<td>Given the Parapet wall is now included in the recommended plan, the design level of 1542 is appropriate.</td>
</tr>
<tr>
<td>Gate Reliability</td>
<td>7</td>
<td>Gate machinery rehab should be part of FWAC per policy, communicate importance under acceptability, robustness and completeness. If gates do not undergo rehab, risk may end up above TR guidelines. Portray risks associated with gates better.</td>
</tr>
<tr>
<td>Remote Operation of Gates</td>
<td>2</td>
<td>Exclude from FWAC &amp; IRRMP, include in recommended plan due to life safety concerns for project personnel (Dam O/T) as supported by the risk assessment.</td>
</tr>
<tr>
<td>Water Control Manual</td>
<td>8</td>
<td>FWAC assumption of 140,000 cfs discharge threshold may be conservative but does not impact DSMS decision. Need better definition of plan during construction and summary of selection of care and diversion plan.</td>
</tr>
<tr>
<td>Risk Assessment</td>
<td>13</td>
<td>Need to better define/understand how debris assumptions affect the risk estimate with respect to overtopping. Risk Cadre to estimate risks for with and without debris blockage in the without parapet wall scenario. Ensure consistency with plots and assumptions. Identify potential vulnerabilities and include in risk register. Memorandum to Nate Snorteland documenting March 2016 meeting. Identify impacts of uncertainty on decision.</td>
</tr>
<tr>
<td>PMF-IDF</td>
<td>9</td>
<td>IDF is the PMF and a waiver will be required for not designing to IDF. Additional documentation to support change in IDF required. Some additional information to increase. Get IDF approved prior to finalizing DSMR. Bulletin 17-C analysis to be completed prior to DSOG. Research 1840 flood. Finish Atlas 14 comparison. Finish evaluation of historic floods.</td>
</tr>
<tr>
<td>Uplift</td>
<td>19</td>
<td>Finalize the details of drainage system and uplift assumptions for the DSMS. A plan will be developed to assess uplift performance in future and use construction data to inform drain design. Develop and document plans in report for uplift assessment and drainage design.</td>
</tr>
<tr>
<td>Geology</td>
<td></td>
<td>More robust discussion of geologic structure is needed. Additional data compilation, analysis, and graphics to be developed during PED.</td>
</tr>
<tr>
<td>RMP 7</td>
<td>3</td>
<td>Screening documentation not adequate to support eliminating but decision to eliminate is correct. Include better documentation of what is included in this plan and better describe why it was screened.</td>
</tr>
<tr>
<td>Training Walls</td>
<td>3</td>
<td>Analysis is conservative since the it neglects the stilling basin and scour protection slabs and loads may be overestimated resulting in anchors that may be overdesigned or even not necessary. Utilize refined loads from future physical modeling to update the design during PED.</td>
</tr>
<tr>
<td>Environmental Issues</td>
<td>4</td>
<td>Mitigation Plan needs more detail; Record of Decision needs to be updated to reflect mitigation commitments. PDT is waiting on input from Fish &amp; Wildlife; Final Mitigation actions will be resolved in PED; will required tiered NEPA documentation.</td>
</tr>
<tr>
<td>Instrumentation</td>
<td>4</td>
<td>Provide more detail and clarifications regarding instrumentation layout and data loggers</td>
</tr>
<tr>
<td>Consequences</td>
<td>5</td>
<td>No significant revisions required.</td>
</tr>
</tbody>
</table>

243 June 2017
An unusual complicating factor for the ATR of this Report was the fact that there were four separate, concurrent reviews ongoing. Besides this ATR there was an Independent External Peer Review (IEPR); a Quality Control and Consistency Review (QCC); and a special Review Panel focused specifically on issues related to drainage and uplift. The objectives of the latter study are provided below:

<table>
<thead>
<tr>
<th>For each component of the drainage system evaluated (and for the drainage system in general), the Review Panel shall address, at a minimum, the following:</th>
</tr>
</thead>
<tbody>
<tr>
<td>• Is the component of the drainage system necessary/required for the design?</td>
</tr>
<tr>
<td>• Does the component of the drainage system reduce risk for the project?</td>
</tr>
<tr>
<td>• Is the component of the drainage system cost effective with consideration given to construction cost and future maintenance costs?</td>
</tr>
<tr>
<td>• Does the component of the drainage system allow for required monitoring? Instrumentation? Maintenance? Future repair?</td>
</tr>
<tr>
<td>• Can the design of the component be optimized to reduce risks and/or cost? If so, how?</td>
</tr>
<tr>
<td>• Are there risks associated with the use of a particular component? If so, can the risks be overcome and how?</td>
</tr>
<tr>
<td>• Does the Panel recommend the installation of the component of the drainage system? Are there components of the system the Panel recommends to remove? Are there additional components the Panel recommends to add/modify/combine?</td>
</tr>
<tr>
<td>• What drain effectiveness/uplift distribution does the Panel recommend be used with this system with consideration given to the TSP design as a whole? Can this recommendation be supported by guidance? What future monitoring/maintenance is required to maintain this recommendation?</td>
</tr>
</tbody>
</table>

The PDT attempted to address each of the issues raised by the special Review Panel, and some of the results of those deliberations resulted in changes to the DSMR that were not a result of the ATR. Coordination of how all these reviews and resolutions of comments merged into the Final Report made the PDT’s task very challenging. A number of the special Review Panel’s recommendations resulted in significant changes to the TSP details in regard to drainage and access to drains for cleanout.

Some of the most challenging issues involved the planned and necessary revisions to the Water Control Manual (WCM) that will be accomplished over the long life of the project. Water management activities were not clearly described and were needed to support the TSP, particularly the scope and schedule for water management actions needed to support the overall project schedule.

Another issue that generated a great deal of discussion in the ATR was the decision whether or not to include construction of Parapet Wall on the dam crest to raise the dam crest elevation. Many features of the RMP-6 were designed to allow the dam to perform for a pool of EL 1,542-ft. However the dam crest is at EL 1,535-ft. The TSP originally included construction of a parapet wall to provide a new dam crest of EL 1,542-ft. To evaluate the way forward on this issue, the Bluestone Risk Cadre performed a risk assessment to specifically assess the impact of construction of the parapet wall. The cadre found that while there is a sizeable reduction in likelihood of breach between pools at EL 1,535-ft and EL 1,546-ft, there is essentially no reduction between EL 1,550-ft and the IDF. Due to the remote likelihood of all these events, the total risk associated with this failure mode is driven by the upper portion of the loading curve. In other words, storm events resulting in EL 1,550-ft and above are driving the risk. This issue was
essentially resolved when the DSOG directed the PDT in November 2016 not to include completion of the Parapet Wall in the final RMP-6.

None of the ATR members found it necessary to provide a statement of findings but confirmed by email that all comments were resolved to their satisfaction.

There are several unresolved issues associated with the ATR; a number of the ATR comments can only be resolved fully during the PED stage, during which further hydraulic model studies will continue alongside development of Plans and Specifications. However the ATR team fully agreed that there was nothing that needed to be resolved in order to issue the Final DSMR.

Comment No. 6672105: Reviewer asked for the following action in this comment: “Reconcile the inconsistency between the report recommendation to eliminate the parapet wall from consideration and the selection of 1542 as the design level for RMP 6.” This generated a long discussion with the PDT that overlapped with the special Review Panel’s recommendations. This resulted in the PDT stating that “The PDT is working with the RMC to define a plan for optimization activities. The plan will identify which items need to be optimized for the DSMS and which items can be optimized during PED. The DSMS report will include documentation of this plan.” However the scope of the RMC’s “optimization plan” was not focused on being completed in time to be a part of the Final Bluestone DSMR. The ATR Team was satisfied that the RMC’s “Optimization Plan” could be provided in a memo to follow issuance of the Final DSMR.

Comment No. 6670185: Reviewer believes the existing second stage stilling basin end sill and baffle blocks could be anchored rather than demolishing them and replacing them with new structures with the same geometry. The PDT stated that since hydraulic model testing is not fully complete, the loads applied to the second stage baffles and end sill are purely theoretical. There is insufficient information regarding the actual expected baffle loads to contemplate an analysis more exhaustive than that performed for the DSMS. Moving forward into PED, the results of modeling will guide the action to be taken with respect to second stage stilling basin features.

Comment Nos. 6671869 and 6671967: Reviewer has requested that a figure be developed showing a profile with all the aspects of the drains, weir flows, groundwater elevations, geology, anchors, grout losses, and plugged or partially plugged drains should be compiled. Although Phase IV anchor installation is not yet complete, this figure could be used to better evaluate the relationship between the grout losses from anchors and performance of the drains or reduction in permeability.

Comment No. 6672082: Reviewer doubts, based on preliminary review of the penstock joint mapping, that a massive sliding planar surface exists that would permit movement of the dam and spillway area on the order of several hundred feet. He also states there is definitely not a N20W trending release plane that can accommodate this and therefore there may be significant 3-D effects that are not accounted for in terms of the geometry of a basal sliding slab. This comment basically questions the fundamental premise driving the need for the dam safety modification – the premise that the spillway monoliths could slide downstream if overtopping-induced erosion removes a significant amount of the passive wedge.

Comment No. 6672083: Reviewer pointed out that the Training Wall modifications and anchoring are based on unrealistic and very conservative hydraulic loading assumptions. Comment response stated “The training wall analysis featured in the DSMS is a conservative analysis of the selected training wall monoliths as stand-alone structures using Phase "A" physical modeling water profiles and high/low tailwater loadings. The conservative water level heights used will be revised and refined as future physical modeling is completed.”

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Comment No. 6672796: Reviewer requests that the pre-anchor installation data for weir flow, uplift cell, drain flow, and piezometer data be plotted vs. reservoir elevation, and then another data set be plotted with the same parameters measured after the completion of the anchoring, to compare the overall drainage and uplift data pre- and post-anchor construction. This data is available, and should be compiled onto graphical format that can show the difference in these conditions before and after anchors or drain rehabilitation efforts.

Comment Nos. 668042, 6668665, 6671683 (and others): Reviewer recommended making numerous changes in Table 1-1, particularly in regard to the Water Control Manual. Reviewer also noted in regard to hydrologic routing that the Main Report text states that existing WCM is assumed which does not include operation of the penstocks, and a referenced Figure shows operation of the penstocks. Although the PDT did make extensive changes to the referenced table, it was noted by the ATR reviewer that the update of the normal water control plan to incorporate use of the penstocks should already be complete, but was not. Therefore update of the normal water control plan to incorporate use of the penstocks needs to be described as an immediate need, needs to be started ASAP, and must be completed no later than 2019.

8.5. Quality Control and Consistency Review
Aaron Smith (Project Manager - LRH), Scott Wheeler (Lead Engineer - LRH), Kim Davidson (Geologist - LRH) and Nick Koutsunis (Hydraulic Engineer - LRH) presented to the Quality Control and Consistency (QCC) review panel on October 12, 2016 in Lakewood, CO at the Risk Management Center. The QCC panel consisted of Larry Von Thun (geotechnical engineer), Steve Higinbotham (hydraulic structures engineer), Scott Jones (structural engineer) and Dan Hertel (constructability and cost estimating). The QCC panel members agreed that all the incremental risks are reasonably portrayed and that the urgency of the actions to address PFM 33 is low. However, there was consensus amongst the panel members that a failure mode at Bluestone Dam that poses unacceptable risk.

The QCC panel concurred on the following tasks moving forward:

- The selected plan to reduce incremental risk associated with PFM33 (spillway monolith instability) should be completed including additional anchors in the dam, modifying the first stage stilling basin to prevent rock scour, reinforcing the second stage basin to minimize scour, and address scour concerns with overtopping of the training walls.
- The ongoing risk analysis to assess the benefit of adding an additional drainage gallery near the toe of the dam should be completed to determine whether the new gallery should be added to the selected plan.
- The following QCC comments should be addressed in the Final DSMR:
  - Summarize additional dam anchor requirements in conjunction with the proposed stilling basin anchors analyzed as a system to confirm the required anchors and drains to reduce risk to an acceptable level.
  - Clarify whether the spillway obstructions (gate, gate frames, bridge deck) are to be removed or retained with the selected plan.
  - Summarize factors where uncertainties are not a factor in the overall decision for the selected plan such as hydrologic loading, three dimensional effects for monolith stability, and erodibility of the unlined basin sections.
  - Add the selected plan total project cost estimate to the report.
- Successfully address all pertinent ATR comments on the report prior to final submittal.
The only dam safety recommendation that resulted from the QCC Meeting was an ALARP evaluation of the parapet wall on the dam crest to reduce non-failure risk during overtopping (PFM33, PFM34 and PFM35).

8.6. Dam Senior Oversight Group (DSOG)
The Huntington District presented the FWAC risk assessment and recommended plan to the Dam Senior Oversight Group (DSOG) in Knoxville, TN on November 1, 2016. The DSOG panel concurred with the DSAC 2 classification, conclusions of the ECRA and FWAC risk assessments, and the recommendation of RMP 6 (modified stilling basin with super-cavitating baffles and no parapet wall). The topics listed in the following bullets were highlights from the discussion at the meeting.

1. There was discussion whether to implement both an underdrain system and a toe gallery, both of which would requireanchoring to achieve required safety factors. DSOG majority supported RMP 6 in 2 phases.
   a. Implement Risk Management Plan 6 (Bluestone Phase 5 – new stilling basin with under drains) to reduce risks from PFM 33.
   b. Implement Bluestone Phase 6 (toe gallery) to reduce risks from PFM 33, only if assumed uplift reduction is not achieved following implementation of Phase 5 with under drains
2. Implementation of the parapet wall in accordance with ALARP to further reduce total risks was not supported by the DSOG as the risk estimate showed no appreciable risk reduction benefit.
3. Toe gallery proposed to be Phase 6 (wait for Phase 5 to validate need). Construction would not be preapproved, rather would hold for further data and evaluation.
4. Provide cost estimate and PMP to the Director of the RMC.
5. Prepare a memorandum for HQ Dam Safety Officer giving endorsement of selected plan.

8.7. Type 1 Independent External Peer Review (IEPR)
The Independent External Peer Review (IEPR) of the Bluestone Dam, Summers County, West Virginia Draft Dam Safety Modification Report is documented in the Comment-Response Record for Final Report and Addendum to Final Report dated 16 February 2017. This is the final product of a working document used by the IEPR Panel and the USACE, and coordinated by APMI, during the comment-response phase of the IEPR for the Bluestone Dam, Draft Dam Safety Modification Report. This document provides Final Panel Comments (FPCs), the USACE’s evaluator responses to the FPCs, and the BackChecks of the Independent External Peer Review (IEPR) panel members. This is a consolidated document and includes all of the comments for the Bluestone Dam IEPR: FPCs #1 through #33 are for the Final Report and FPCs #34 through #36 are for the Addendum to the Final Report. This report can be found in Appendix P (Review Documentation).

8.8. Quality Assurance by Major Subordinate Command (MSC)
This review is still underway.

8.9. Headquarters Office of Water Project Review (OWPR)
This review is still underway.

Public comments were solicited during public scoping and in response to the SDEIS. The public scoping comments along with a SDEIS comments/response matrix that was prepared can found in Appendix A of the SFEIS.

Comments received during the public scoping meeting included:
• Two anonymous comments were received from the public during the public scoping meeting. One comment was oral and was regarding construction traffic using roadways through Bellepoint. The second comment was written and was regarding the impact on the community from prolonged construction, noise and dust, and replacement of the ballpark in Bellepoint which had been removed and the space utilized for construction staging during the ongoing dam modifications.

• Four written comments were received from agencies including City of Charleston, NPS, West Virginia Division of Culture and History, and West Virginia Department of Natural Resources (WVDNR).

Public and agency comments received during the SDEIS public review and comment period and responses to those comments are provided in a comment matrix in Appendix A of the SFEIS. Revisions to the SEIS were required, based on comments, and the comment matrix identifies the areas of the SEIS that were revised. A total of 60 comments encompassing 22 general categories were recorded. Table 8-3 describes the number of comments received for each general category.

Table 8-3: SDEIS Public Review and Public Hearing Comments

<table>
<thead>
<tr>
<th>Category Number</th>
<th>General Category of Comments</th>
<th>Number of Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Agency Involvement/Coordination</td>
<td>10</td>
</tr>
<tr>
<td>2</td>
<td>Inundation Impacts Upstream</td>
<td>10</td>
</tr>
<tr>
<td>3</td>
<td>Mitigation Sites, Measures, or Plan</td>
<td>5</td>
</tr>
<tr>
<td>4</td>
<td>T&amp;E Species or wildlife/plants impacts</td>
<td>4</td>
</tr>
<tr>
<td>5</td>
<td>Air Quality Impacts or Climate Change</td>
<td>4</td>
</tr>
<tr>
<td>6</td>
<td>Increased Sedimentation</td>
<td>3</td>
</tr>
<tr>
<td>7</td>
<td>Resource Values</td>
<td>3</td>
</tr>
<tr>
<td>8</td>
<td>Water Quality or Contamination</td>
<td>3</td>
</tr>
<tr>
<td>9</td>
<td>Project Support</td>
<td>3</td>
</tr>
<tr>
<td>10</td>
<td>Alternatives</td>
<td>2</td>
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9. References


### 10. Bluestone DSMR Acronym List

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<tr>
<th>Acronym</th>
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<tr>
<td>AALL</td>
<td>Average Annual Life Loss</td>
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<td>ACE</td>
<td>Annual Chance of Exceedance</td>
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<td>ALARP</td>
<td>As Low As Reasonably Practicable</td>
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<tr>
<td>APF</td>
<td>Annual Probability of Failure</td>
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<tr>
<td>BCRA</td>
<td>Baseline Condition Risk Assessment</td>
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<td>CAP</td>
<td>Continuing Authorities Program</td>
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<td>CE</td>
<td>Constructability Evaluation</td>
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<td>CELRD</td>
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<td>cfs</td>
<td>cubic feet per second</td>
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<td>CG</td>
<td>Construction General</td>
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<td>CSRA</td>
<td>Cost and Schedule Risk Assessment</td>
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<td>Construction Work Limits</td>
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<td>CY</td>
<td>Cubic Yards</td>
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<td>DAMRAE</td>
<td>Dam Risk Analysis Engine Software</td>
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<tr>
<td>DDR</td>
<td>Design Documentation Report</td>
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<td>DQC</td>
<td>District Quality Control</td>
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<td>DSMR</td>
<td>Dam Safety Modification Report</td>
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<tr>
<td>DSOG</td>
<td>Dam Senior Oversight Group</td>
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<td>EA</td>
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<td>Future Without Federal Action Condition</td>
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<td>Flood Warning and Emergency Evacuation Plan</td>
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<td>FWO</td>
<td>Future Without</td>
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<td>GI</td>
<td>General Investigation</td>
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<td>Geographic Information System</td>
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<tr>
<td>HEC-FIA</td>
<td>Hydrologic Engineering Center Flood Impact Analysis Software</td>
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</tbody>
</table>
HPU – Hydraulic Power Units
HQ - Headquarters
HTRW – Hazardous, Toxic, and Radioactive Waste
IDF – Inflow Design Flood
LEDPA – Least Environmentally Damaging Practicable Alternative
LRD – Great Lakes and Ohio Rivers Division
LRH – Huntington District
LRL – Louisville District
IRRM – Interim Risk Reduction Measures
MCC – Motor Control Center
MCE – Maximum Credible Earthquake
MFR – Memorandum for Record
MH – Maximum High
MSL – Mean Sea Level
NAVD - North American Vertical Datum
NED – National Economic Development
NEPA – National Environmental Policy Act
NGVD – National Geodetic Vertical Datum
NPS – National Park Service
NWI – National Wetlands Inventory
NWS – National Weather Service
NWW – Northwest Walla Walla
OBE – Operating Basis Earthquake
OMBIL – Operation and Maintenance Business Information Link
OMRR&R – Operations, Maintenance, Repair, Replacement and Rehabilitation
OSE – Other Social Effects
P&G – Principles and Guidelines
PA – Periodic Assessment
PAL – Planning Aid Letter
PAR – Population At Risk
PAS – Planning Assistance to States
PDT – Project Delivery Team
PED – Preconstruction Engineering and Design
PFM – Potential Failure Mode
PFMA – Potential Failure Mode Analysis
PGA – Peak Ground Acceleration
PI – Periodic Inspection
PLC – Programmable Logic Controller
PMF – Probable Maximum Flood
PMP – Probable Maximum Precipitation
PTI – Post Tensioning Institute
PUP – Probability of Unsatisfactory Performance
RED – Regional Economic Development
RMC – Risk Management Center
RMP – Risk Management Plans
S&A – Supervision and Administration
SDF – Spillway Design Flood
SDEIS – Supplemental Draft Environmental Impact Statement
SEIS – Supplemental Environmental Impact Statement
SFEIS – Supplemental Final Environmental Impact Statement
SPRA – Screening Portfolio Risk Assessment
SRP – System Response Probabilities
T&E – Threatened and Endangered Species
TRG – Tolerable Risk Guidelines
TSP – Tentatively Selected Plan
USBR – United State Bureau of Reclamation
USFWS – United States Fish and Wildlife Service
UST – Underground Storage Tank
USACE – United States Army Corps of Engineers
VE – Value Engineering
WEA – Wireless Emergency Alert
WCM – Water Control Manual
WCP – Water Control Plan