US Army Corps of Engineers ®
Walla Walla District

MILL CREEK FLOOD RISK MANAGEMENT GENERAL INVESTIGATION

Draft Feasibility Study and Environmental Assessment Report

January 2020

APPENDIX A
HYDROLOGY AND HYDRAULICS

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MILL CREEK FLOOD RISK MANAGEMENT GENERAL INVESTIGATION FEASIBILITY STUDY AND ENVIRONMENTAL ASSESSMENT REPORT – DRAFT APPENDIX A, HYDROLOGY AND HYDRAULICS

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ANNEX

Annex, Climate Change Summary

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SECTION 1 - INTRODUCTION

The hydrology and hydraulics work began with looking into ways to narrow down the large number of alternatives remaining after the AMM screening to those with increasing likelihood of being the Tentative Selected Plan (TSP). The analysis focused on using existing and readily available tools and information to accomplish three rounds of screening.

Initial screening looked at project trade-offs, to determine the most cost effective alternative to meet the same target flood risk protection. This analysis did not look at economics. Analysis was conducted using a rough MATLAB based tool that assessed the proposed projects impacts to flows and flow frequencies. Flow frequencies relied on the existing 2008 regulated flood frequency curve in the Mill Creek Water Control Manual. This rough concept design information was shared with the other engineering disciplines which determined quantities and costs. The initial list of Trade-off alternatives is shown in Table 1-1.

Table 1-1. Trade-off Alternatives

Project Tradeoff	Project Alternative	Flood Risk Target[1]
New Project	New Large Dam	0.1-Percent (1,000 year)
	New Large Bypass Channel	0.1-Percent (1,000 year)
	New Small Dam	0.1-Percent (1,000 year)
	New Small Bypass Channel	0.1-Percent (1,000 year)
Rehab Concrete Channel verses a new project	Rehab	0.5-percent (200 year)
	New Dam	0.5-percent (200 year)
	New Bypass Channel	0.5-percent (200 year)
Small Modification to Existing Project	Small Dam Raise	0.43-Percent (230 year)
	Small Excavation	0.43-Percent (230 year)
	Small Levee Raise	0.43-Percent (230 year)
Moderate Modification to Existing Project	Moderate Dam Raise	0.37-Percent (270 year)
	Moderate Excavation	0.37-Percent (270 year)
	Moderate Levee Raise	0.37-Percent (270 year)
Large Modification to Existing Project	Large Dam Raise	0.32-Percent (310 year)
	Large Excavation	0.32-Percent (310 year)
	Large Levee Raise	0.32-Percent (310 year)
Levee Setback verses Levee Raise	Levee Setback verses Raise	varies

[1] Frequency based on 2008 regulated frequency curve in the Mill Creek water control manual

The second round of screening looked at the maximum benefit analysis. A roughly built Dam Safety Modification Study (DSMS) based HEC-RAS model was used to determine the 0.1 percent flood inundation area with existing conditions and project operations. The inundation area was provided to economics to determine the maximum benefit value to identify any project alternatives that should not be carried forward due to costs likely exceeding benefits. This was used to screen out alternatives too costly to meet a maximum benefit to cost ratio.

Based on the screening results of the Trade-off analysis, and Maximum Benefit Analysis more detailed analysis was conducted on the remaining alternatives. In addition, a combination alternative was added which included a Small Excavation and Small Levee Raise, and an operations change alternative was added. For this analysis H&H models were developed using software from the Hydrologic Engineering Center (HEC). The specific models utilized were HEC-SSP, HEC-WAT, HEC-ResSim, HEC-RAS, and HEC-FDA. These were 80% models, but adequate for determining the TSP. HEC-SSP was utilized for generating the new unregulated frequency curves. The ResSim model was used to evaluate reservoir operations for existing conditions as well as the alternatives, the HEC-WAT model was used for random sampling of inflow hydrology, routing it through ResSim and generating the regulated flow frequency curve data. The HEC-RAS model was used for hydraulic routing of the hydrographs to determine inundation areas associated with the alternatives. And the HEC-FDA model was used for levee sizing. Using detailed analysis described above, the following alternatives shown in Table 1-2 were evaluated.

Table 1-2. Full Modeled Alternatives

Project Alternative	Project Flow Objective (cubic feet per second)	Flood Risk in Design Channel For Flow Objective (Percent AEP)[2]
Existing Conditions[1]	3,500 cfs through levees	0.57-Percent (175 Year)
Small Excavation	3,500 cfs through levees	0.51-Percent (196 Year)
Moderate Excavation	3,500 cfs through levees	0.40-Percent (250 Year)
Small Levee Raise	3,700 cfs through levees	0.50-Percent (200 Year)
Moderate Levee Raise	4,000 cfs through levees	0.40-Percent (250 Year)
Small Excavation and Small Levee Raise[1]	3,700 cfs through levees	0.42-Percent (238 Year)
Operational Change – Increased Initial Regulated Flow[1]	3,500 cfs through levees	0.39-Percent (255 Year)

[1]Alternative was not in the trade-off analysis
[2]Frequency based on updated regulated frequency curve for this study

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SECTION 2 - TRADE-OFF ANALYSIS AND MAXIMUM BENEFITS ASSESSMENT

2.1 MATLAB SCREENING TOOL DEVELOPMENT

Due to the large number of potential alternatives, a tool was needed that could analyze alternatives without consuming significant computational or user time. Yet this tool also needed to be robust enough to screen the selected trade-off alternatives with reliable outcomes. A code was written to achieve this using the MATLAB (matrix laboratory) programming language.

The most recent regulated peak discharge frequency curve in the water control manual for Mill Creek (2008) was originally developed by first scaling up the historic 1996 unregulated flood hydrograph (using a constant scale factor) to match the peak discharge on the unregulated frequency curve. This was done several times for a range probabilities, with a maximum scale factor of 1.77 for the 0.2-percent (500-yr) event. This resulted in multiple hydrographs, each having a peak corresponding to a different unregulated probability. Then these hydrographs were regulated using the flood control rule curve from the water control manual. This same process was implemented in MATLAB for the present study, but the code was written to be flexible such that storage elevation curves, rule curve inflection points, initial pool elevation, and maximum pool elevation could be changed quickly. This allowed the user to quickly iterate to optimize alternatives (runtime is about 0.14 seconds per event; 73 events are run). In addition to a regulated frequency curve, the program output includes regulated hydrographs for visual analysis. To facilitate this flexibility, an equation was used in place of the actual rule curve. That equation was used to plot the flood control rule curve shown in Figure 2-1.

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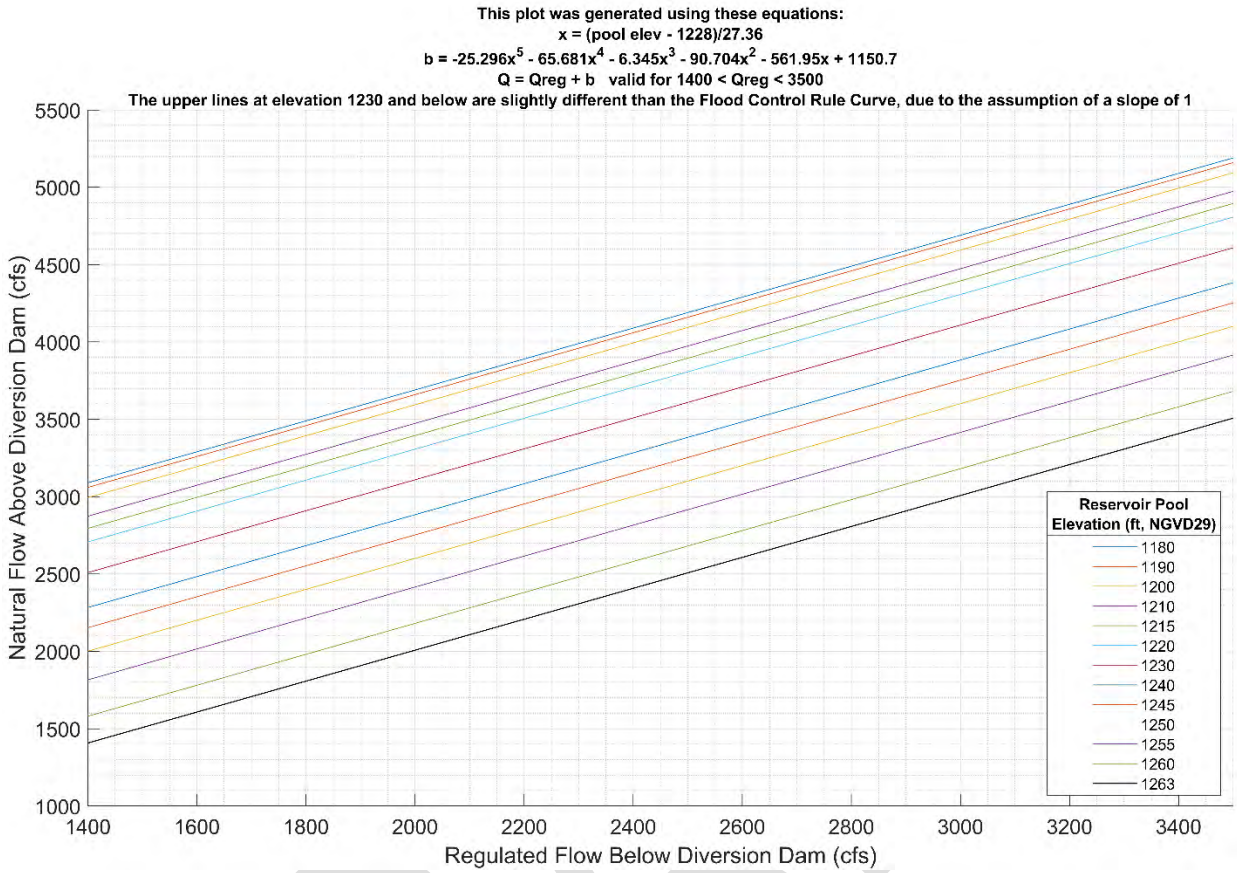

Figure 2-1. Equation Based Flood Control Rule Curve Used to Estimate the Water Control Manual Rule Curve

As with the current regulated flood frequency curve, a discrete number of hydrographs were run through the MATLAB model. For convenience and consistency the hydrographs and unregulated frequency curve from the previous study were used, plus one additional 1,000 yr return period hydrograph (requiring a scale factor of 2.15), resulting in a total of 73 hydrographs. Two major limitations to the MATLAB analysis were 1) the small number of probabilities (73 events) considered and 2) the use of only a single hydrograph shape. Using a single hydrograph shape scaled to a peak value imposes an implicit assumption that the hydrograph is balanced in terms of probability across durations (e.g., the 1-hr peak has the same probability as the 20-hr peak) and will remain balanced after scaling, which is not precisely true. A balanced hydrograph could have been used instead, which could have been scaled in a balanced fashion, but this was not done because it would have resulted in an improbable shape, introducing a different set of oversimplifications; regulation can be sensitive to hydrograph shape.

During the screening process these assumptions were kept in mind; and very similar alternatives were not eliminated using this tool. The two limitations listed above would be eliminated later with the full HEC-WAT model, which would use multiple hydrograph shapes and a large event sample size. The final HEC-WAT model showed sufficiently similar results to

the MATLAB analysis to validate its previous use as a screening tool, and to demonstrate that the two limitations did not affect which alternative was selected.

2.2 MODELING OUTCOMES

2.2.1 New Project Trade-Off

The first trade-off leading to TSP was to determine the most cost effective way to provide the Walla Walla Valley 0.1-percent level of flood risk management via a single large project. The project could be either a large dam at some location upstream the existing Diversion Dam on Mill Creek, or it could be a by-pass channel that would route excess floodwaters around the community. This task was completed prior to detailed Hydrology and Hydraulic models being available, and used flood frequency values from the Mill Creek Water Control Manual, Plate 7-1 dated September 2008. Order of magnitude (Rough) models were created using MATLAB. Two different project sizes were evaluated.

The larger of the two project sizes was a design intended to provide 0.1-percent flood risk management, with a maximum flow of 1,400 cfs through the community. This flow would alleviate damaging high water downstream of the existing levee reach, and allow for community modification and usage of the concrete channel through the center of the city. This would require a storage dam be constructed with an active reservoir capacity of 29,000 acre-feet. Operations assumed all flow passes the new reservoir and Bennington Lake up to 1,400 cfs, then flows over that amount would be stored. An equivalent by-pass channel could be constructed rather than the new storage reservoir. An equivalent bypass channel would be designed to immediately take runoff over 1,400 cfs. When the bypass flows exceed 4,500 cfs (combined flow of 5,900 cfs), Bennington Lake would store the balance up to the 0.1-percent flood. For this operation we would not use the existing rule curves, but would divert to Bennington Lake when the combined bypass and channel reach 5,900 cfs.

The smaller of the two project sizes is a design intended to provide 0.1-percent flood risk management, with a maximum flow of 3,500 cfs through the community. This design would maintain the same level of maximum design flood flows as the levee/concrete channel experience in present operations, but with increased flood risk reduction to the 0.1-percent flood. This would require a storage dam be constructed with an active reservoir capacity of 12,800 acre-feet. Operations assumed all flow passes the new reservoir and Bennington Lake up to 1,400 cfs, then flows stored would follow the existing rule curve for Bennington Lake up to the downstream levee/concrete channel design flow of 3,500 cfs. An equivalent bypass channel could be constructed rather than a new storage reservoir. An equivalent bypass channel would be designed to immediately take runoff over 1,400 cfs, and the rule curve for Bennington Lake would be modified to pass flows through the existing levee/concrete channel reach up to 3,500 cfs after the bypass channel capacity is exceeded. For this operation the new bypass channel would need to carry 2,600 cfs. Table 2-1 summarizes the pertinent information.

Table 2-1. New Project Trade-Off

	Design Flood Risk Management	New Storage (acre-feet)	New By-Pass Channel Capacity (cfs)	New By-Pass Channel Average Cross-Section Area (sq ft) / depth (ft)*	Flow through levee/concrete channel reach (cfs)
New Large Dam	0.1-Percent (1,000 year)	29,000	N/A	N/A	1,400
New Large Bypass Channel	0.1-Percent (1,000 year)	N/A	4,700	332 / 7	1,400
New Small Dam	0.1-Percent (1,000 year)	12,800	N/A	N/A	3,500
New Small Bypass Channel	0.1-Percent (1,000 year)	N/A	2,600	224 / 5	3,500

*Cross section area assumes a 0.0113 ft/ft average slope, a trapezoidal channel with 1:1 side slopes and a 40 foot wide bottom, with a friction value of 0.035. Probabilities were computed using the MATLAB screening tool.

2.2.2 New Project or Rehab the Existing Channel Trade-off

This trade-off analysis leading to the TSP is to look at the most cost effective way of maintaining our current level of flood risk reduction, ~ 0.5-percent. The trade-off is rehabbing portions of the concrete channel or constructing a new project to mitigate the reliable flow through town. This task was completed prior to detailed Hydrology and Hydraulic models being available, and used flood frequency values from the Mill Creek Water Control Manual, Plate 7-1 dated September 2008. Order of magnitude (Rough) models were created using MATLAB. For this analysis we look at rehab costs to maintain the present channel reliability, or building an upstream project to limit flows to the channel bottom of the concrete channel (assumed to be 1,400 cfs).

No change to the present flood risk reduction capacity of the flood control project would be made for the rehab alternative. For the new project mitigation alternative, no channel rehabilitation would be made, and flows would be limited to 1,400 cfs for the 0.5-percent flood. One option would be to construct a new storage dam to mitigate. This would require a storage dam be constructed with an active reservoir capacity of 11,000 acre-feet. Operations assumed all flow passes the new reservoir and Bennington Lake up to 1,400 cfs, then flows over that amount would be stored. An equivalent by-pass channel could be constructed rather than the new storage reservoir. An equivalent bypass channel would be designed to immediately take runoff over 1,400 cfs. When the bypass flows exceed 1,800 cfs (combined flow of 3,200 cfs), Bennington Lake would store the balance. For this operation we would not use the existing rule curves, but would divert to Bennington Lake when the combined bypass and channel reach

3,200 cfs. For this reason, the combined capacity for channel flow can be less that the rehab options that continues to use the existing rule curve. Table 2-2 summarizes the pertinent information.

Table 2-2. Rehab verses Mitigation Project Trade-off

	Design Flood Risk Management	New Storage (acre-feet)	New By-Pass Channel Capacity (cfs)	New By-Pass Channel Average Cross-Section Area (sq ft) / depth (ft)*	Flow through levee/concrete channel reach (cfs)
Rehab	0.5-percent (200 year)	N/A	N/A	N/A	3,500
New Dam	0.5-percent (200 year)	11,000	N/A	N/A	1,400
New Bypass Channel	0.5-percent (200 year)	N/A	1,800	174 / 4	1,400

*Cross section area assumes a 0.0113 ft/ft average slope, a trapezoidal channel with 1:1 side slopes and a 40 foot wide bottom, with a friction value of 0.035. Probabilities were computed using the MATLAB screening tool.

2.2.3 Small Modification to Existing Project - Trade-off.

This task is to determine the most cost effective way to provide a minor increase in flood risk reduction. The options looked at were a Levee Raise, a Dam Raise, or Excavation in the reservoir pool. This task was completed prior to detailed Hydrology and Hydraulic models being available, and used flood frequency values from the Mill Creek Water Control Manual, Plate 7-1 dated September 2008. Order of magnitude (Rough) models were created using MATLAB.

The comparison were made starting with a 2 foot dam raise. This would involve raising Bennington Lake Dam and the Diversion Dam as well as associated structures. The 2 foot dam raise was determined as a 2 foot increase in the pool maximum water surface elevation of 1,265 feet NGVD29. The actual heights of the dam may need to be raised more or less than 2 feet depending on settlement or changes in design guidance. Using the rough models created using MATLAB, the equivalent excavation volume to equate to a 2 foot dam raise was determined, as well as the increase in regulated flow that would offset the dam raise. The HEC-RAS model (under development) was used to determine the height of levee raise required to handle the increased regulated flow. The levee raise was based on the increase in the water surface resulting from an increase in the maximum levee reach flow of 3,500 cfs to the new design release. In this way the dam raise, excavation, and levee raise all would provide the same flood risk reduction benefit, in this case increasing flood risk reduction from ~0.5-percent chance exceedance (200 year) to 0.43-percent chance exceedance (230 year). The levee raise assumes that the stabilizers can handle increased flows without modification. It should be noted that this is not a full risk based analysis and actual needed levee heights to meet FEMA

Accreditation requirements or Corps Risk Based guidelines are not yet assessed. In addition there are areas with less than 2 feet of freeboard with the 3,500 cfs flow rate and may not meet FEMA Accreditation requirements in their present condition. Table 2-3 summarizes the pertinent information.

Table 2-3. Small Modification to Existing Project Trade-Off

	Design Flood Risk Management	Dam Raise (ft)	Reservoir Excavation (acre-feet)	Rise in Water Surface compared to 3500 cfs (ft)	Levee Raise (ft) to get 3 foot freeboard	Flow through levee/concrete channel reach (cfs)
Small Dam Raise	0.43-Percent (230 year)	2	N/A	N/A	N/A	3,500
Small Excavation	0.43-Percent (230 year)	N/A	462	N/A	N/A	3,500
Small Levee Raise	0.43-Percent (230 year)	N/A	N/A	0.4	0 – 2.2 depending on location	3,700

*Probabilities were computed using the MATLAB screening tool.

2.2.4 Moderate Modification to Existing Project - Trade-Off

This task is to determine the most cost effective way to provide a moderate increase in flood risk reduction. The options looked at were a Levee Raise, a Dam Raise, or Excavation in the reservoir pool. This task was completed prior to detailed Hydrology and Hydraulic models being available, and used flood frequency values from the Mill Creek Water Control Manual, Plate 7-1 dated September 2008. Order of magnitude (Rough) models were created using MATLAB.

The comparison were made starting with a 5 foot dam raise. This would involve raising Bennington Lake Dam and the Diversion Dam as well as associated structures. The 5 foot dam raise was determined as a 5 foot increase in the pool maximum water surface elevation of 1,265 NGVD29. The actual heights of the dam may need to be raised more or less than 5 feet depending on settlement or changes in design guidance. Using the rough models created using MATLAB, the equivalent excavation volume to equate to a 5 foot dam raise was determined, as well as the increase in regulated flow that would offset the dam raise. The HEC-RAS model (under development) was used to determine the height of levee raise required to handle the increased regulated flow. The levee raise was based on the increase in the water surface resulting from an increase in the maximum levee reach flow of 3,500 cfs to the new design release. In this way the dam raise, excavation, and levee raise all would provide the same flood risk reduction benefit, in this case increasing flood risk reduction from 0.5-percent chance exceedance (200 year) to 0.37-percent chance exceedance (270 year). The levee raise assumes

that the stabilizers can handle increased flows without modification. It should be noted that this is not a full risk based analysis and actual needed levee heights to meet FEMA Accreditation requirements or Corps Risk Based guidelines are not yet assessed. In addition there are areas with less than 2 feet of freeboard with the 3,500 cfs flow rate and may not meet FEMA Accreditation requirements in their present condition. Table 2-4 summarizes the pertinent information.

Table 2-4. Moderate Modification to Existing Project Trade-Off

	Design Flood Risk Management	Dam Raise (ft)	Reservoir Excavation (acre-feet)	Rise in Water Surface compared to 3500 cfs (ft)	Levee Raise (ft) to get 3 foot freeboard	Flow through levee/concrete channel reach (cfs)
Moderate Dam Raise	0.37-Percent (270 year)	5	N/A	N/A	N/A	3,500
Moderate Excavation	0.37-Percent (270 year)	N/A	1,188	N/A	N/A	3,500
Moderate Levee Raise	0.37-Percent (270 year)	N/A	N/A	0.75	0 – 2.5 depending on location	4,000

*Probabilities were computed using the MATLAB screening tool.

2.2.5 Large Modification to Existing Project - Trade-Off

This task is to determine the most cost effective way to provide a larger increase in flood risk reduction. The options looked at were a Levee Raise, a Dam Raise, or Excavation in the reservoir pool. This task was completed prior to detailed Hydrology and Hydraulic models being available, and used flood frequency values from the Mill Creek Water Control Manual, Plate 7-1 dated September 2008. Order of magnitude (Rough) models were created using MATLAB.

The comparison were made starting with the maximum Excavation in the Bennington Reservoir of 2,209 acre feet. The equivalent dam raise would be 9 feet, involving raising Bennington Lake Dam and the Diversion Dam as well as associated structures. The 9 foot dam raise was determined as a 9 foot increase in the pool maximum water surface elevation of 1,265. The actual heights of the dam may need to be raised more or less than 9 feet depending on settlement or changes in design guidance. Using the rough models created using MATLAB, the equivalent increase in regulated flow was determined that would offset the dam raise or excavation. The HEC-RAS model (under development) was used to determine the height of levee raise required to handle the increased regulated flow. The levee raise was based on the increase in the water surface resulting from an increase in the maximum levee reach flow of 3,500 cfs to the new design release. In this way the dam raise, excavation, and levee raise all

provide the same flood risk reduction benefit, in this case increasing flood risk reduction from 0.5-percent chance exceedance (200 year) to 0.32-percent chance exceedance (310 year). The levee raise assumes that the stabilizers can handle increased flows without modification. It should be noted that this is not a full risk based analysis and actual needed levee heights to meet FEMA Accreditation requirements or Corps Risk Based guidelines are not yet assessed. In addition there are areas with less than 2 feet of freeboard with the 3,500 cfs flow rate and may not meet FEMA Accreditation requirements in their present condition. Table 2-5 summarizes the pertinent information.

Table 2-5. Large Modification to Existing Project Trade-Off

	Design Flood Risk Management	Dam Raise (ft)	Reservoir Excavation (acre-feet)	Rise in Water Surface compared to 3500 cfs (ft)	Levee Raise (ft)	Flow through levee/concrete channel reach (cfs)
Large Dam Raise	0.32-Percent (310 year)	9	N/A	N/A	N/A	3,500
Large Excavation	0.32-Percent (310 year)	N/A	2,209	N/A	N/A	3,500
Large Levee Raise	0.32-Percent (310 year)	N/A	N/A	1.2	0 – 2.7 depending on location	4,300

*Probabilities were computed using the MATLAB screening tool.

2.2.6 Levee Setback Project

Set back levees provide opportunities to increase habitat for land, air, and water wildlife. They also provide for more natural stream function with pooling areas and wetlands. Due to developed land constraints there are no reaches of adequate length to obtain a measurable reduction in flood stages by the use of setback levees, when increased friction resulting from habitat plantings is accounted for. However, these features could be added with no adverse impact to the existing levee reaches of the Mill Creek Flood Control Project if appropriately designed. There are three proposed areas for levee set back. The first location is on the south side of the Mill Creek channel extending from Diversion dam to private property. The second location is on the north side of the Mill Creek channel extending from Diversion dam to private property.

Figure 2-2 - Levee Setback Locations in Federal Reach

The third location is the Mill Creek Sports complex located downstream of Tausick Way.

Figure 2-3 - Levee Setback Locations in Sports Complex

Levee Setbacks should be sized to have consistent hydraulic height to the levees they are replacing. If being compared to the existing condition, the levee setback would offer no benefit. If being compared to a levee raise, the levee setback costs could be offset by the cost that would have been incurred by the levee raise, however, the setback levee would need the added height.

2.3 MAXIMUM BENEFIT HEC-RAS MODEL FOR THE 0.1 PERCENT CHANCE FLOODPLAIN

A HEC-RAS model existed for the study reach at the beginning of this study. This model was developed by a former Dam Safety Modification Study. The model relied on as-built drawings for bridges and utilized LiDAR for cross sections and 2D areas. This model was utilized to determine the 0.1 Percent Chance Floodplain. That floodplain was used for determining the maximum project benefits, a ceiling to measure how much project could be afforded. The model was developed with 5 scenarios. These were: No Breach/No Blockage, Tunnel Blockage, Clinton Street Bridge Failure, Levee Failure upstream Tausick Bridge, and Levee Failure upstream the Division Works. The results of this model were passed to the economist to determine the maximum project benefit, which would be used to screen out more costly alternatives.

2.4 CONCLUSION AFTER GEOTECHNICAL AND COST ENGINEERING ANALYSIS

Note: The following conclusions were not drawn solely from hydrology and hydraulics analysis but from the project development team efforts as a whole. They are included here to inform the reader of the study process that led to the next hydrology and hydraulics section.

The new project tradeoff for a new large dam or bypass channel and for a new small dam or bypass channel showed that the bypass channel was consistently more cost effective than a new dam for obtaining the same benefits.

The channel rehabilitation tradeoff for a new mitigation project showed it was more cost effective to rehabilitate the existing channel than to build a project to mitigate.

The tradeoff analysis for the small, medium and large scaled enhancement to the existing project included: dam raise, levee raise, or excavation from the dam pool area. This tradeoff analysis showed that the dam raise consistently was more expensive than the other two alternatives for decreasing flood risk.

Levee set back reaches were identified and found to be too short to offer any flood risk reduction benefits.

The maximum benefits, as defined as the 0.1 percent chance flood, showed that benefits are not great enough to support a new project or a large change to an existing project.

After this analysis what remained for the next stage of the analysis is:

- Channel Rehab
- Small Excavation
- Small Levee Raise
- Moderate Excavation
- Moderate Levee Raise
- Operational Changes (not included in the tradeoff analysis as it is a no cost change

SECTION 3 - FULL MODELED ALTERNATIVES

Alternatives that were not screened out during the trade-off analysis, or had costs that exceeded the benefits afforded by 0.1-percent chance flood protection, were further investigated by full modeling.

3.1 OVERVIEW AND ALTERNATIVE DESCRIPTIONS

This effort began with development of Unregulated Flow Frequency Curves using HEC-SSP. These curves were input into a HEC-WAT model coupled with HEC-ResSim model for random sampling to develop regulated flow frequency curves for Existing Conditions and each project alternative. The resulting regulated peak frequencies were utilized in an unsteady flow HEC-RAS 1D/2D model to generate inundation mapping sets for existing conditions and each alternative. The alternatives evaluated are shown in Table 3-1.

Table 3-1. Full Modeled Project Alternatives

Project Alternative	Project Flow Objective (cubic feet per second)	Flood Risk Target in Design Channel For Flow Objective (Percent AEP)
Existing Conditions	3,500 cfs through levees	0.57-Percent (175 Year)
Small Excavation	3,500 cfs through levees	0.51-Percent (196 Year)
Moderate Excavation	3,500 cfs through levees	0.40-Percent (250 Year)
Small Levee Raise	3,700 cfs through levees	0.50-Percent (200 Year)
Moderate Levee Raise	4,000 cfs through levees	0.40-Percent (250 Year)
Small Excavation and Small Levee Raise	3,700 cfs through levees	0.42-Percent (238 Year)
Operational Change – Increased Initial Regulated Flow	3,500 cfs through levees	0.39-Percent (255 Year)

The following are the alternative descriptions:

<u>Existing Conditions</u>

This is the base alternative. This alternative utilizes all existing facilities and operations plans that are present today. An unregulated inflow Frequency curve was developed in HEC-SSP and used for this and all other project alternatives. The HEC-WAT model was utilized to randomly sample unregulated inflow events and route them through the HEC-ResSim model to create regulated outflows. The HEC-ResSim model is an enhanced version of the DSMS model. The elevation-storage table used in the DSMS model was based on a survey conducted in 1983, so a new elevation-storage table was produced using a LiDAR terrain set from 2011. A regulated outflow frequency curve was developed.

Nine hydrographs, representing specific regulated flow frequencies were routed through the RAS model to create inundation areas. The RAS model was an enhanced version of the DSMS model that now includes as built data of the levee channel and concrete channel geometries. In the DSMS model these had been estimated using LiDAR. The model was developed to run with 5 scenarios. These included:

1. Channel functions as designed.
2. A 90% blockage of the tunnel portion of the concrete channel due to building collapse
3. A 50% blockage of the Otis Street Bridge due to upstream wall failure
4. Overtopping failure of the levee upstream the Tausick Way Bridge
5. Overtopping failure of the levee upstream the Yellowhawk/Garrison Division works.

The resulting inundation areas were provided to the economist for use in HEC-FDA.

<u>Small Excavation</u>

For this alternative 370 acre-feet was removed from the reservoir floor adding to the storage volume available. The excavation was taken above pool elevation 1,205 NGVD29 (conservation pool) and below elevation 1,265 NGVD29 (full pool). The small excavation option was originally 462 acre-feet in the trade-off analysis but was scaled down to 370 acre-feet as that was all that could be removed effectively for the knob in the middle of the lake without excavating above full pool, elevation 1,265 ft (NGVD 29), which would have no effect at storing water during a flood. The HEC-WAT model was utilized to randomly sample unregulated inflow events and route them through the HEC-ResSim model to create regulated outflows. The elevation-storage curve in the HEC-ResSim model was modified to include the excavation, making the additional storage useable when the unregulated inflows exceed the maximum regulated flow, 3500 cfs. A regulated outflow frequency curve was developed.

Eight hydrographs, representing specific regulated flow frequencies were routed through the RAS model to create inundation areas. The RAS model was an enhanced version of the DSMS model that now includes as built data of the levee channel and concrete channel geometries. In the DSMS model these had been estimated from LiDAR. The model was developed to run with 5 scenarios. These included:

1. Channel functions as designed.
2. A 90% blockage of the tunnel portion of the concrete channel due to building collapse
3. A 50% blockage of the Otis Street Bridge due to upstream wall failure
4. Overtopping failure of the levee upstream the Tausick Way Bridge
5. Overtopping failure of the levee upstream the Yellowhawk/Garrison Division works.

The resulting inundation areas were provided to the economist for use in HEC-FDA.

Moderate Excavation

For this alternative 1,188 acre-feet was removed from the reservoir floor adding to the storage volume available. The excavation was taken above pool elevation 1,205 NGVD29 (conservation pool) and below elevation 1,265 NGVD29 (full pool). The HEC-WAT model was utilized to randomly sample unregulated inflow events and route them through the HEC-ResSim model to create regulated outflows. The elevation-storage curve in the HEC-ResSim model was modified to include the excavation, making the additional storage useable when the unregulated inflows exceed the maximum regulated flow, 3,500 cfs. A regulated outflow frequency curve was developed.

Eight hydrographs, representing specific regulated flow frequencies were routed through the RAS model to create inundation areas. The RAS model was an enhanced version of the DSMS model that now includes as built data of the levee channel and concrete channel geometries. In the DSMS model these had been estimated from LiDAR. The model was developed to run with 5 scenarios. These included:

1. Channel functions as designed.
2. A 90% blockage of the tunnel portion of the concrete channel due to building collapse
3. A 50% blockage of the Otis Street Bridge due to upstream wall failure
4. Overtopping failure of the levee upstream the Tausick Way Bridge
5. Overtopping failure of the levee upstream the Yellowhawk/Garrison Division works.

The resulting inundation areas were provided to the economist for use in HEC-FDA.

Small Levee Raise

For this alternative the maximum design flow for the project was increased from 3,500 cfs to 3,700 cfs. The design flow also coincides with the 1-percent chance flood. The levees were raised such that they could pass a 3,700 cfs 1 percent chance flood and be certifiable for the National Flood Insurance Program. The criteria was from ECB 2019-11 (EC 1165-2-218 Draft).

The WAT model was utilized to randomly sample unregulated inflow events and route them through the HEC-ResSim model to create regulated outflows. The table referenced by the diversion rule was changed to increase the maximum regulated flow from 3,500 cfs to 3,700 cfs in the HEC-ResSim model. Below 3,500 cfs, the operations should be identical to what would occur in the Existing Conditions model. A regulated outflow frequency curve was developed.

Nine hydrographs, representing specific regulated flow frequencies were routed through the RAS model to create inundation areas. The RAS model was an enhanced version of the DSMS model that now includes as-built data of the levee channel and concrete channel geometries. Levees were raised in this model to pass 3,700 cfs with 85-percent conditional non-exceedance probability. No changes to the concrete channel were evaluated as its design was for flows in excess of this amount. The model was developed to run with 5 scenarios. These included:

1. Channel functions as designed.
2. A 90% blockage of the tunnel portion of the concrete channel due to building collapse
3. A 50% blockage of the Otis Street Bridge due to upstream wall failure
4. Overtopping failure of the levee upstream the Tausick Way Bridge
5. Overtopping failure of the levee upstream the Yellowhawk/Garrison Division works.

The resulting inundation areas were provided to the economist for use in HEC-FDA.

Moderate Levee Raise

For this alternative the maximum design flow for the project was increased from 3,500 cfs to 4,000 cfs. The design flow also coincides with the 1-percent chance flood. The levees were raised such that they could pass a 4,000 cfs 1-percent chance flood and be certifiable for the National Flood Insurance Program. The criteria was from ECB 2019-11 (EC 1165-2-218 Draft).

The HEC-WAT model was utilized to randomly sample unregulated inflow events and route them through the HEC-ResSim model to create regulated outflows. The table referenced by the diversion rule was changed to increase the maximum regulated flow from 3,500 cfs to 4,000 cfs in the HEC-ResSim model. Below 3,500 cfs, the operations should be identical to what would occur in the Existing Conditions model. A regulated outflow frequency curve was developed.

Eight hydrographs, representing specific regulated flow frequencies were routed through the RAS model to create inundation areas. The RAS model was an enhanced version of the DSMS model that now includes as-built data of the levee channel and concrete channel geometries. Levees were raised in this model to pass 4,000 cfs with 85-percent conditional non-exceedance probability. No changes to the concrete channel were evaluated, as its design was for flows in excess of this amount. The model was developed to run with 5 scenarios. These included:

1. Channel functions as designed.
2. A 90% blockage of the tunnel portion of the concrete channel due to building collapse
3. A 50% blockage of the Otis Street Bridge due to upstream wall failure
4. Overtopping failure of the levee upstream the Tausick Way Bridge
5. Overtopping failure of the levee upstream the Yellowhawk/Garrison Division works.

The resulting inundation areas were provided to the economist for use in HEC-FDA.

Small Excavation and Small Levee Raise

For this alternative 370 acre-feet was removed from the reservoir floor adding to the storage volume available. The excavation was taken above pool elevation 1,205 NGVD29 (conservation pool) and below elevation 1,265 NGVD29 (full pool). The small excavation option was originally 462 acre-feet in the trade-off analysis but was scaled down to 370 acre-feet as that was all that could be removed effectively for the knob in the middle of the lake without excavating above full pool, elevation 1,265 ft (NGVD 29), which would have no effect at storing water during a flood. For this alternative the maximum design flow for the project was increased from 3,500

cfs to 3,700 cfs. The design flow also coincides with the 1-percent chance flood. The levees were raised such that they could pass a 3,700 cfs 1-percent chance flood and be certifiable for the National Flood Insurance Program. The criteria utilized was from ECB 2019-11 (EC 1165-2-218 Draft).

The HEC-WAT model was utilized to randomly sample unregulated inflow events and route them through the HEC-ResSim model to create regulated outflows. The table referenced by the diversion rule was changed to increase the maximum regulated flow from 3,500 cfs to 3,700 cfs in the HEC-ResSim model. The elevation-storage curve in the HEC-ResSim model was modified to include the excavation, making the additional storage useable when the unregulated inflows exceed the maximum regulated flow, 3,700 cfs. A regulated outflow frequency curve was developed.

Nine hydrographs, representing specific regulated flow frequencies were routed through the RAS model to create inundation areas. The RAS model was an enhanced version of the DSMS model that now includes as-built data of the levee channel and concrete channel geometries. Levees were raised in this model to pass 3,700 cfs with 85-percent conditional non-exceedance probability. No changes to the concrete channel were evaluated. The model was developed to run with 5 scenarios. These included:

1. Channel functions as designed.
2. A 90% blockage of the tunnel portion of the concrete channel due to building collapse
3. A 50% blockage of the Otis Street Bridge due to upstream wall failure
4. Overtopping failure of the levee upstream the Tausick Way Bridge
5. Overtopping failure of the levee upstream the Yellowhawk/Garrison Division works.

The resulting inundation areas were provided to the economist for use in HEC-FDA.

<u>Operational Change – Increase Initial Regulated Flow</u>

For this alternative the initial regulated flow was increased from 1,400 cfs to 2,500 cfs. The HEC-WAT model was utilized to randomly sample unregulated inflow events and route them through the HEC-ResSim model to create regulated outflows. The table referenced by the diversion rule was changed to increase the initial regulated flow. A regulated outflow frequency curve was developed. No inundation mapping was developed for this alternative at this time.

Nine hydrographs, representing specific regulated flow frequencies were routed through the RAS model to create inundation areas. The RAS model was an enhanced version of the DSMS model that now includes as-built data of the levee channel and concrete channel geometries. No changes to the concrete channel were evaluated. The model was developed to run with 5 scenarios. These included:

1. Channel functions as designed.
2. A 90% blockage of the tunnel portion of the concrete channel due to building collapse
3. A 50% blockage of the Otis Street Bridge due to upstream wall failure
4. Overtopping failure of the levee upstream the Tausick Way Bridge
5. Overtopping failure of the levee upstream the Yellowhawk/Garrison Division works.

The resulting inundation areas were provided to the economist for use in HEC-FDA.

3.2 REGULATED FLOW FREQUENCY CURVE DEVELOPMENT

The regulated flow frequency curves for each alternative were developed by regulating 50,000 randomly sampled unregulated hydrographs. Each regulated frequency curve is a plot of the 50,000 regulated peaks at their corresponding probabilities, using Weibull plotting positions (Figure 3-27). The random sampling was done using the HEC-WAT program and the hydrographs were regulated using HEC-ResSim, which was run via the HEC-WAT.

3.2.1 Unregulated Frequency Curve Development

As a prerequisite to developing frequency curves, unregulated flows were developed for this study. Figure 3-1 shows a schematic of the Mill Creek Project. Approximately 1.6 miles upstream of the Diversion Dam there is a USGS gage that has been in operation for only about 20 years. That gage is thought to be less accurate at high discharges. For this reason, the more reliable downstream gage (shown in Figure 3-1), which has the longer period of record was used. As a result, regulation needed to be removed from the flows. The unregulated flows could have been computed by adding the diversions to the gaged flow and subtracting the return flows. However, since flows are generally not released from Bennington Lake during a peak flow event, it was only necessary to sum the Intake Canal flows, the Yellowhawk/Garrision diversion, and the USGS gaged flows. Since the Yellowhawk/Garrision diversion records were somewhat sporadic and not available before 1965, a constant 30 cfs discharge was assumed, except during large events when flow was being diverted into Bennington Lake, during which time the project logbook data were used to estimate the actual diversion. The project logbook was also used to estimate the Intake Canal flow.

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Figure 3-1. Schematic of the Mill Creek Project

Volume-duration and peak frequency curves were developed using the Hydrologic Engineering Center's Statistical Software Package (HEC-SSP). A Bulletin 17C analysis was performed for the following durations: peak, 1-hour, 20-hour, 30-hour, 40-hour, and 70-hour. HEC-SSP version 2.1.1 does not provide a volume-duration frequency analysis option using Bulletin 17C (it only does peak flow analysis), so peak flows for the durations listed above were pre-computed before performing the 17C analysis.

The statistics (Table 3-2) from the 1-hour peak unregulated flow frequency curve were input into HEC-WAT for hydrograph sampling. This frequency curve was selected because the hydrograph shape sets are hourly, so it was more appropriate to scale them to an hourly frequency curve than to a peak frequency curve. This curve was developed from sub-hourly flow data, but where sub-hourly data were not available the peak flow was used. As a result, this frequency curve has the same period of record as the peak flow frequency curve, which was a factor in selecting this duration, as discussed in the "

Selecting the Frequency Curve and Scaling **Duration**" section below.

Table 3-2. Unregulated Peak 1-Hour Frequency Curve Statistics

Parameter	Value
Mean (of log)	3.05
Standard deviation (of log)	0.277
Skew (of log)	0.314

Selecting the Frequency Curve and Scaling Duration

Ideally, hydrographs should be scaled to a volume-duration frequency curve, rather than a peak frequency curve, because the ability to regulate a hydrograph is a function of volume rather than peak flow. For the existing operations, the portion of the hydrograph that gets regulated is the portion with discharge exceeding 1,400 cfs. The alternatives evaluated aim to regulate flows with probabilities greater than 0.005 (200 year return period) to 3,500 cfs. To accurately assess regulation for flows in this probability space a range of frequency curve durations should be evaluated, the longest of which is equal to the duration of the portion of the 200 year hydrograph that exceeds 1,400 cfs. But this duration varies based on hydrograph shape. In addition, the scaling duration (frequency curve duration) should not be longer than the duration of the hydrograph being scaled. The shortest duration hydrograph selected as a shape set for scaling is 40 hours. As a result, this duration was selected even though some hydrographs were as much as a few days longer. It should be pointed out that the other, perhaps more relevant, duration is the duration of the portion of the 200 year hydrograph that exceeds 3,500 cfs. That duration was less than 40 hours for half of the hydrographs and only one hydrograph had scaled flows greater than 3,500 cfs for more than 51 hours.

Initially, the unregulated hydrographs were scaled up based on the 40 hour duration (i.e. the hydrographs were scaled over their full durations by a constant scale factor, but the peak 40 hour segment was checked against the frequency curve volume), then the 20 hour and 70 hour durations were plotted for comparison. All three durations compared well with the unregulated HEC-SSP frequency curves, indicating that scaling to any duration in that range would produce similar results. However, the peak flow frequency curve did not match and was slightly too low. For this study the peak flow frequency curve is thought to be the most accurate curve due to a longer period of record for peak flows. The continuous period of record for peak flows is 78 years, with extreme event peaks being recorded for the past 113 years. Sub-daily measurements, on the other hand, have been recorded for the past 56 years and daily measurements were recorded for the 22 years preceding.

Using a daily average measurement (e.g. the 22-year period) to compute a volume-duration frequency curve for durations less than 24 hours would clearly underestimate the flow, but perhaps less obvious is the fact that using a daily average to compute a peak daily (or even a 30 or 40 hour) flow frequency curve also results in an underestimate since the averaging window is fixed (e.g. midnight to midnight), rather than centered around the peak 24 hr period.

Therefore, for the 22-year period of the record where only daily averages and peak flows were available, the flows were adjusted slightly by developing relationships using the daily and peak flow measurements to estimate peak-duration flows for the range of durations less than 48 hours.

If the relevant flow duration for regulating hydrographs is actually closer to 40 hours, then by scaling to a peak (or hourly) frequency curve the implied assumption is that the hydrograph shapes are sufficiently balanced to result in a correct 40-hour frequency curve. This assumption was verified by generating inflow hydrograph frequency curves using the HEC-WAT sampled hydrographs for both the peak and 40-hour durations. The HEC-SSP 40hr curve and the HEC-WAT 40hr curve are roughly the same until about the 80 year return period. Beyond that point the HEC-WAT curve becomes more conservative but still follows the same general trend and remains well within the confidence bounds. Large floods in Mill Creek are dominated by rainfall runoff but there is also a snowmelt component. It is therefore not difficult to imagine a situation, such as heavy rain on frozen ground with snowpack, which could cause the upper end of the frequency curve to bend upward. As a result, given the limited period of record, it seemed prudent to apply the frequency curve that resulted in a more conservative HEC-WAT generated 40 hour frequency curve, namely the HEC-SSP 1hr peak curve.

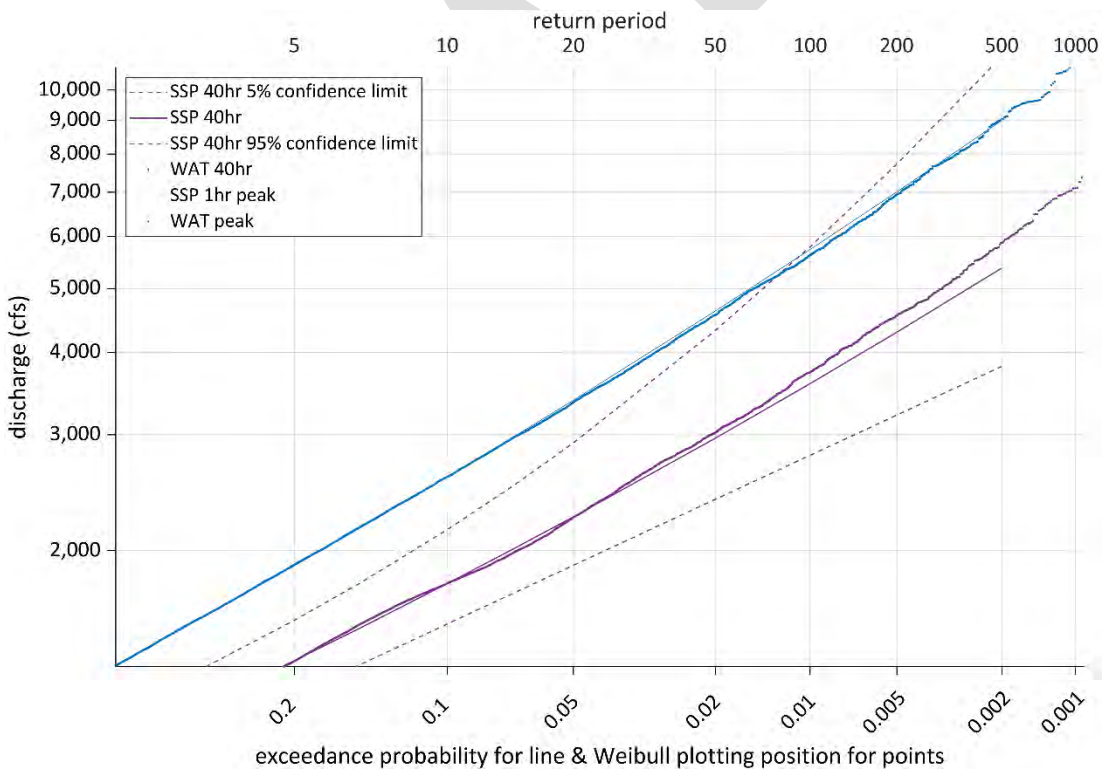

Figure 3-2. HEC-WAT Output Frequency Curves (Scaling to Peak 1hr Curve) Compared to HEC-SSP Curves

3.2.2 HEC-ResSim Model Development

The HEC-ResSim model used for the Mill Creek GI study to examine impact of the various alternatives with modified reservoir operations to take advantage of structural changes was modified from a previous model used for the Mill Creek CWMS Implementation and the Mill Creek Dam Safety Modification Study (DSMS). The Mill Creek DSMS Model had previous been implemented as a HEC-ResSim/HEC-WAT model for simulation of many extreme events with the existing infrastructure and two operational alternatives. This model was adapted to simulate the operations that would be allowed under the various alternatives under consideration for the GI Study.

3.2.2.1 Changes to the DSMS Model to Support GI Study Alternatives

The elevation-storage table used in the DSMS model was based on a survey done in 1983, so a new elevation-storage table was produced using a LiDAR terrain set from 2011. Below the water surface when this was taken (1193 ft NAVD88), the existing curve was interpolated in. This elevation-storage table showed little change from the previous one.

Table 7-1 in the Water Control Manual which determines the diversions from the Diversion Dam into Bennington Lake is interpreted by a rule in the model that looks up the elevation in Bennington Lake and the unregulated flow to calculate the regulated flow downstream and the diversion into the Lake. This rule was modified to reference in an external DSS file based on the name of the HEC-ResSim alternative. For the existing conditions model, this was left un-changed, with points on each line added by interpolation to support modeling the other alternatives.

For each of the alternatives, Table 7-1 was modified using the spreadsheet called "ExtendingRuleCurve.xlsx" to change various parameters. This spreadsheet is capable of changing the initial regulated flow, the maximum regulated flow, and the method used for spacing of the lines on the curve. The spreadsheet outputs a table in a format that can be used by the model and easily copied into the HEC-DSS file referenced by the Table 7-1 rule.

The HEC-ResSim model had previously been verified using the 1996 water year event and was checked using this event to ensure the model was responding appropriately for the existing conditions and each of the alternatives.

3.2.2.2 Existing Conditions

The existing conditions alternative was used as described in the "Changes to the DSMS model to support GI study alternatives" section 3.2.2.1. This was modeled using the HEC-ResSim alternative "FC_WCM"

3.2.2.3 Levee Raise Alternatives

For the levee raise alternatives, the table referenced by the diversion rule was changed to increase the maximum regulated flow from 3,500 cubic feet per second (cfs) to either 3,700 cfs

or 4,000 cfs for the two sizes of levee raise under consideration. The lines on Table 7-1 were extended by extrapolation to the point where their maximum is either 3,700 or 4,000 cfs. Below 3,500 cfs, the operations should be identical to that which would occur in the Existing Conditions model. These alternatives were modeled in HEC-ResSim under the alternative names "LR3700" and "LR4000".

3.2.2.4 Excavation Alternatives

The excavation alternative required several additional steps to model: adjusting the HEC-ResSim model's physical parameters and deciding how to best use the newly available storage. First, the HEC-ResSim model had to be changed to include an elevation-storage curve for Bennington Lake to represent each of the excavation options. A table of soil volume excavated in 5-ft bands was provided by GIS, which was added to the existing conditions elevation-storage curve in 1-ft increments over the range for which excavation occurred. Excavation was limited to above 1,205 NGVD29 (1208 NAVD88 in the data provided) which is the conservation pool, below which flood space would not be available. The excavation amounts provided by GIS ended up being slightly different than the total modeled in the previous MATLAB modeling. The small excavation option which was originally 462 acre-feet was scaled down to 370 acre-feet as that was all that could be removed effectively for the knob in the middle of the lake without excavating above full pool, elevation 1,265 ft (NGVD 29), which would have no effect at storing water during a flood. The moderate option considered additional areas for excavation and was able to reach close to the full volume requested of 1,188 acre-feet. As the GIS excavation analysis was approximate (reaching a total of 1,104 acre-feet) to save time in computing the volume of soil, the values for each tier were scaled up to reach the total of 1,188 acre-feet.

The second problem is that the existing Table 7-1 operation is tied to elevation of water in Bennington Lake, not volume of water stored. Several methods were considered for allocating the additional storage, but it was decided to model two of them: allocation by storage and allocation by percent of total storage. In the allocation by storage method, the new Table 7-1 elevations were assigned by adjusting the line to the elevation that presents the same volume of storage as was available in existing conditions. This method showed the most promise for regulating the largest of events, as it only uses the additional storage when the unregulated inflows exceed the maximum regulated flow, 3,500 cfs for the Small Excavation and Moderate Excavation alternatives.

The allocation by percent of total storage was also examined to maximize the use of the reservoir across the entire range of regulated flows. This method would further reduce the regulated flow from above 1,400 cfs compared to the Existing Conditions alternative, but would have be less effective at regulating the largest of events. This method was modeled in HEC-ResSim but not considered further as it had less effect on decreasing flood flows for large events.

These alternative were modeled in HEC-ResSim under the model alternative "EXSMStor" for the small excavation alternative and "EXMEStor" for the moderate excavation alternative.

3.2.2.5 Combined Small Levee Raise and Small Excavation Alternative

The combined levee raise and excavation alternative used the small excavation alternative's elevation-storage curve, but with the addition of a 3,700 cfs maximum regulated flow. This alternative was modeled in HEC-ResSim under the model alternative "EXSMLR37".

3.2.2.6 Operational Change - Increased Initial Regulated Flow Alternative

For the alternative modeling an operation change with an increased initial regulated flow from 1,400 cfs to 2,500 cfs, the ExtendingRuleCurve.xlsx spreadsheet was used to adjust the initial regulated flow. This results in a reduction in flood flows as the reservoir is not storing water until higher flows are reached and has additional capacity.

The Mill Creek Project is authorized to prevent flood damage in Walla Walla and vicinity. The 1,400 to 2,500 cfs flow range can cause significant impacts downstream the design channel on Mill Creek. When flows go above 1,400 cfs the natural channel changes from a relatively fixed bed to a mobile bed. Observations during the February 1996 flood suggest significant impacts downstream of the City of Walla Walla, with channel migration causing significant agricultural lands being made inaccessible by damage to privately maintained roads, outbuildings being washed away, and flood fight efforts required to prevent the channel from undercutting the foundations of homes and other structures. Current methods of modeling flood damages do not capture erosion and sediment movement related damages. Current methods for estimating damages are based on depth of water over an unchanging terrain model considering only damages to structures and crops, not the loss of land or improvements. While there may be decreases in peak flows during the low frequency events with this alternative, increasing the initial diversion will increase these difficult to account for damages from frequent events. For this alternative to move forward, the economics modeling needs to be revised to account for erosion and mobile bed related damages.

3.2.3 HEC-WAT Model Development

The Flood Risk Analysis (FRA) compute option in the HEC-WAT (Hydrologic Engineering Center Watershed Analysis Tool) software was used to perform risk analysis using a Monte Carlo simulation. The HEC-WAT links HEC-ResSim to the Hydrologic Sampler when performing this simulation. For this study, the Hydrologic Sampler was used to randomly select hydrograph shapes and scale them to random probabilities. These hydrographs are then used as input into the HEC-ResSim model. This random sampling incorporates natural variability into the model. The sampling is correlated to the inflow (unregulated) frequency curve statistics from HEC-SSP (Table 3-2). As a result, if peak inflow frequency curve statistics from HEC-SSP are input into the Hydrologic Sampler and an extremely large number of hydrographs are sampled then the resulting output (unregulated) peak frequency curve will match the inflow frequency curve. However, the frequency curve input by the user has uncertainty associated with it because it was developed from observed flow data taken from a limited period of record. This uncertainty, termed knowledge uncertainty, can optionally be accounted for using a nested Monte Carlo sampling structure. In this nested structure, flows are randomly sampled based on

the number of years in the period of record, generating a series of new frequency curves that represent curves that might have been observed during a different period of record. This process accounts for knowledge uncertainty, and the final output can be subdivided into a family of frequency curves representing the frequency curve uncertainty. Using this uncertainty feature in FRA requires a much larger number of sampled events. As a result, for the TPS analysis we did not turn on the knowledge uncertainty feature in the hydrologic sampler. Furthermore, this uncertainty cannot be readily incorporated into HEC-FDA.

Hydrograph Shape Sets

A hydrograph shape set is a number of hydrographs that get randomly sampled during an HEC-WAT run, and scaled up (or down) based on a randomly sampled probability. The shape sets are predefined by the user and should represent the range of expected hydrograph shapes for the basin. The shape sets are assigned probability ranges over which they can be randomly selected. These probabilities need to be selected judiciously, such that the distribution of hydrographs shapes is properly represented over the probability space. To avoid arbitrary judgment, all hydrographs available over a certain flow threshold (discussed below) were used and assigned an equal probability so that the sample distribution would be the same as the observed distribution of hydrograph shapes. However, some judgment was made about the smallest and largest hydrographs, which were restricted to the low and high flow range of the frequency curve. A screen shot from HEC-WAT in Figure 3-3 shows the range over which hydrograph shape sets could be sampled. Note that the critical range for regulation, which is the range with probabilities less than roughly 0.005 (200 year return period), always has at least 10 possible hydrograph shapes (12 shapes for probabilities greater than 0.004).

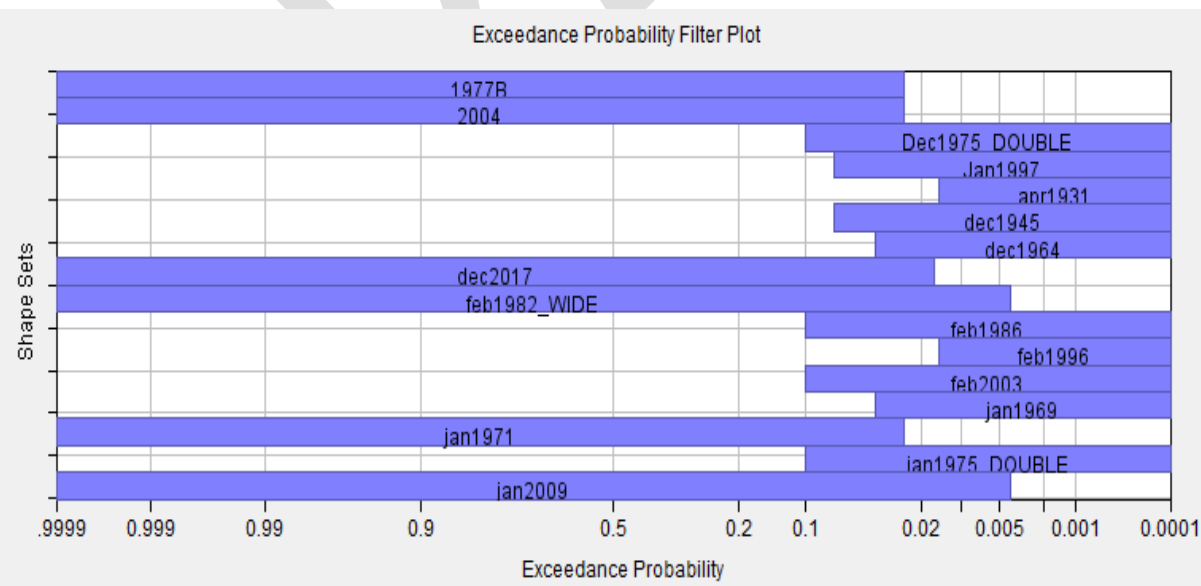

Figure 3-3. Probability Ranges for Shape Sets Used in the Hydrologic Sampler

All events in the period of record that had peaks greater than 1,500 cfs *and* were the largest event during their water year in terms of either the peak, 20, 30, or 40 hour duration were selected as shape sets. The rational for this was that only these events contributed to the

upper half of the frequency curve, so selecting other events (even if they were large enough to scale) would have resulted in a different distribution of shapes than was used for the frequency curve. A handful of events could not be included because their hydrograph shapes were not well defined, sometimes because they occurred during the period of record without hourly (or even daily) data. However, it was possible to use some events from this period because their shapes had been documented after the floods (1931, 1945, and Dec 1964). This resulted in a total of 16 hydrograph shapes depicted in Figure 3-4**.** These shapes are also plotted by season in Figure 3-5.

The shape sets have been modified slightly from the historic events by subtracting a constant value of between 9 and 89 cfs, depending on the event. This was done to prevent over scaling the baseflow. Modifying the shape sets made the baseflow approximately correct when the hydrographs were scaled up to about the 200-yr return period, which is near the critical return period for regulation. Shape sets that are not sampled over that range were not adjusted. During scaling, a constant scale factor is applied to the entire shape set. Table 3-3 shows a range of scale factors for each hydrograph shape, which were computed prior to baseflow adjustment (i.e. based on the historic peaks).

Many of the scale factors for the 500-yr and 1000-yr return periods are somewhat uncomfortably large, which increases the uncertainty in the upper end of the frequency curve.

Table 3-3. Scale Factors for Selected Probabilities

	Probability:	**0.01**	**0.005**	**0.002**	**0.001**
	Return Period:	**100 yr**	**200 yr**	**500 yr**	**1000 yr**
Shape	**Water Year**	**Scale Factor**			
Mar/Apr 1931	1931	0.97	1.19	1.52	1.82
Dec 1945	1946	1.89	2.31	2.96	3.55
Dec 1964	1965	1.74	2.12	2.73	3.26
Jan 1969	1969	1.75	2.14	2.75	3.29
Jan 1971	1971	-	-	-	-
Jan 1975	1975	2.41	2.94	3.78	4.52
Dec 1975	1976	2.20	2.69	3.45	4.13
Dec 1977	1978	-	-	-	-
Feb 1982	1982	2.82	3.45	-	-
Feb 1986	1986	2.16	2.65	3.40	4.06
Feb 1996	1996	0.85	1.04	1.34	1.60
Dec 1996 - Jan 1997	1997	1.91	2.34	3.00	3.59
Jan/Feb 2003	2003	2.40	2.93	3.76	4.50
Jan 2004	2004	-	-	-	-
Jan 2009	2009	2.82	3.45	-	-
Dec 2017	2018	-	-	-	-

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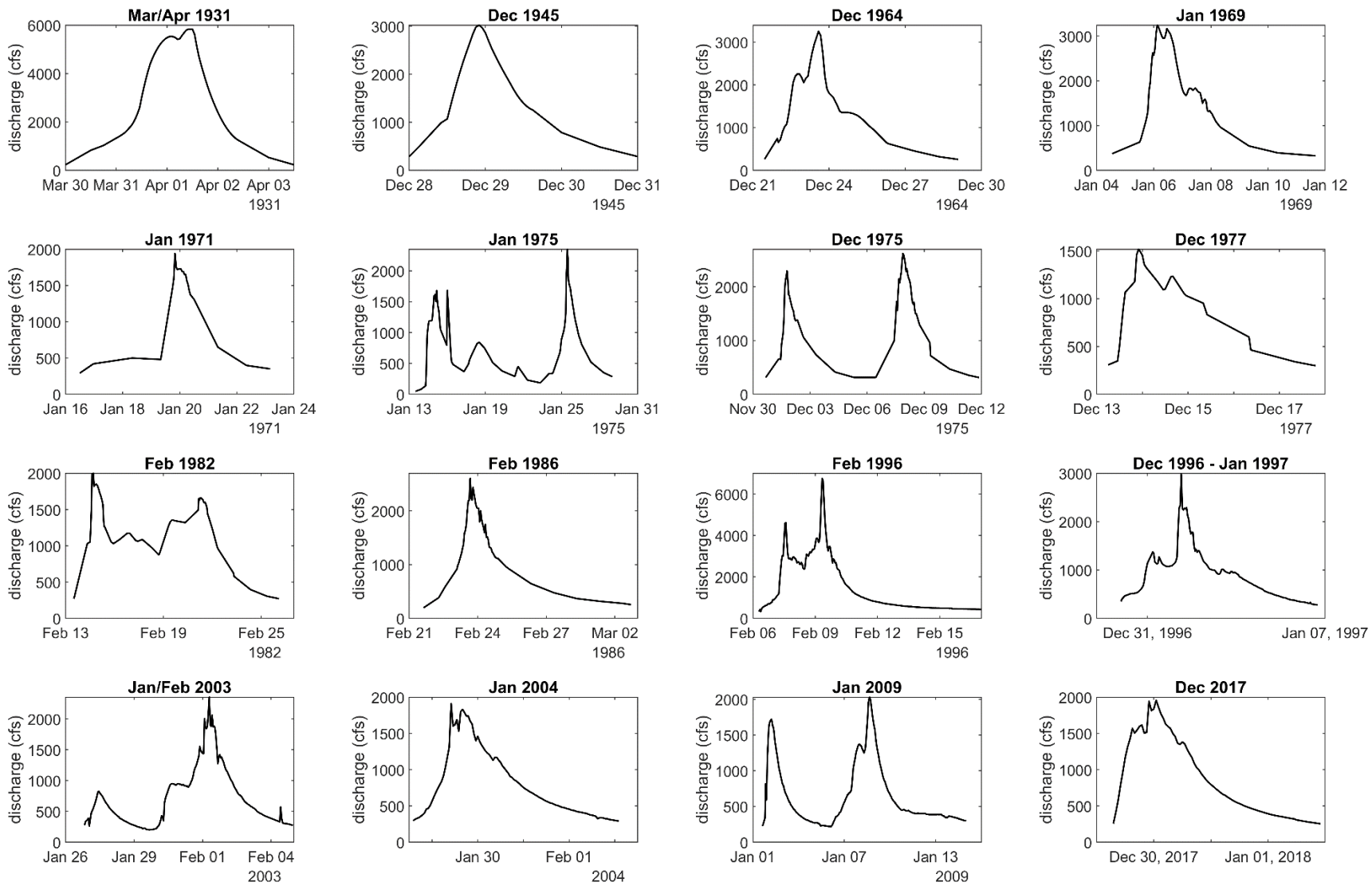

Figure 3-4. Hydrograph Shape Set

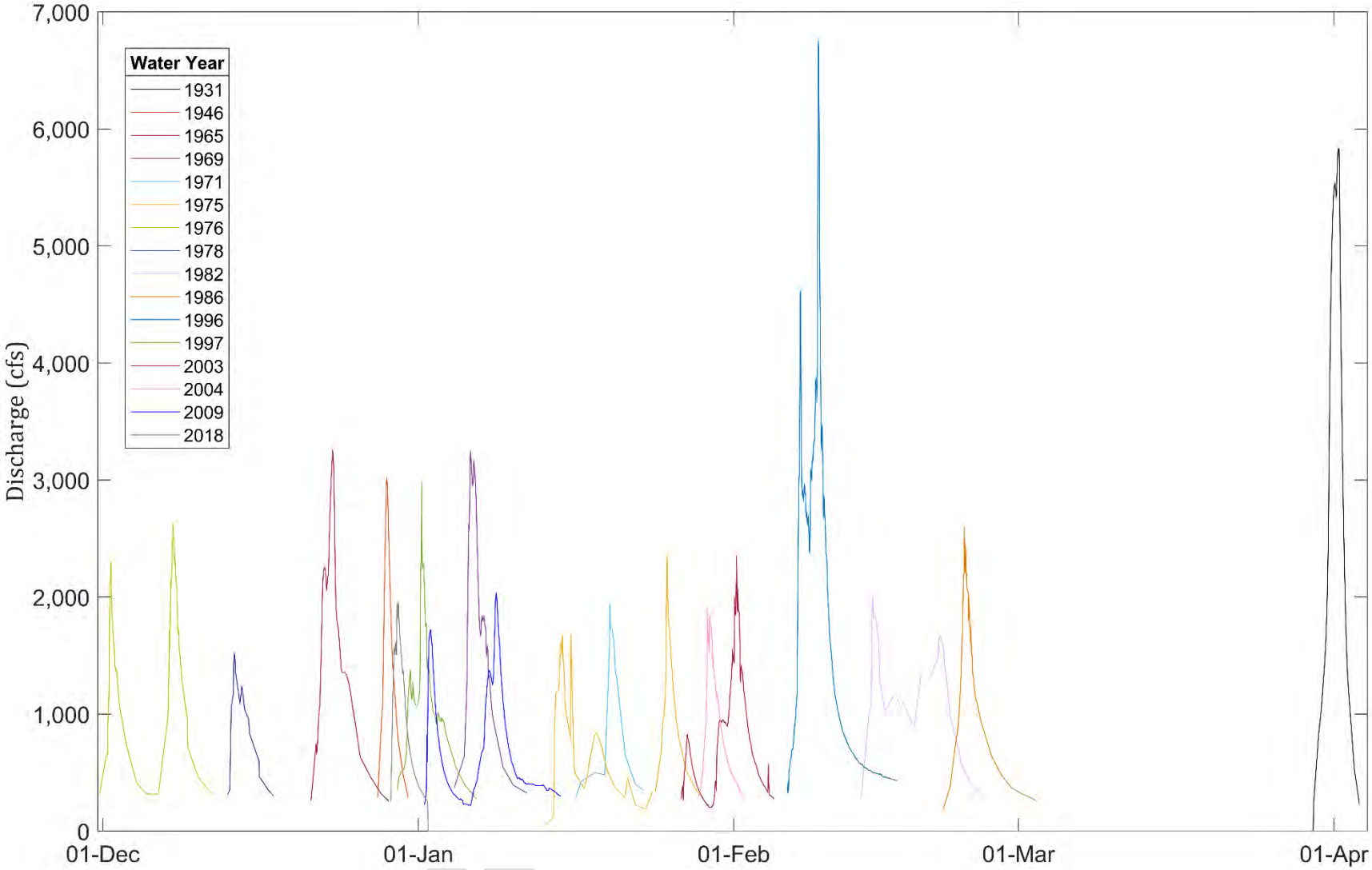

Figure 3-5. Hydrograph Shapes by Season

3.2.4 Regulated Frequency Curves for Existing Conditions and Alternatives

The output from the HEC-WAT model is the regulated flow frequency curves. These are presented for existing conditions and for each alternative as well as how they are input in HEC-FDA as follows:

<u>Existing Conditions</u>

The regulated flow frequency curve from HEC-WAT output is shown in Figure 3-6.

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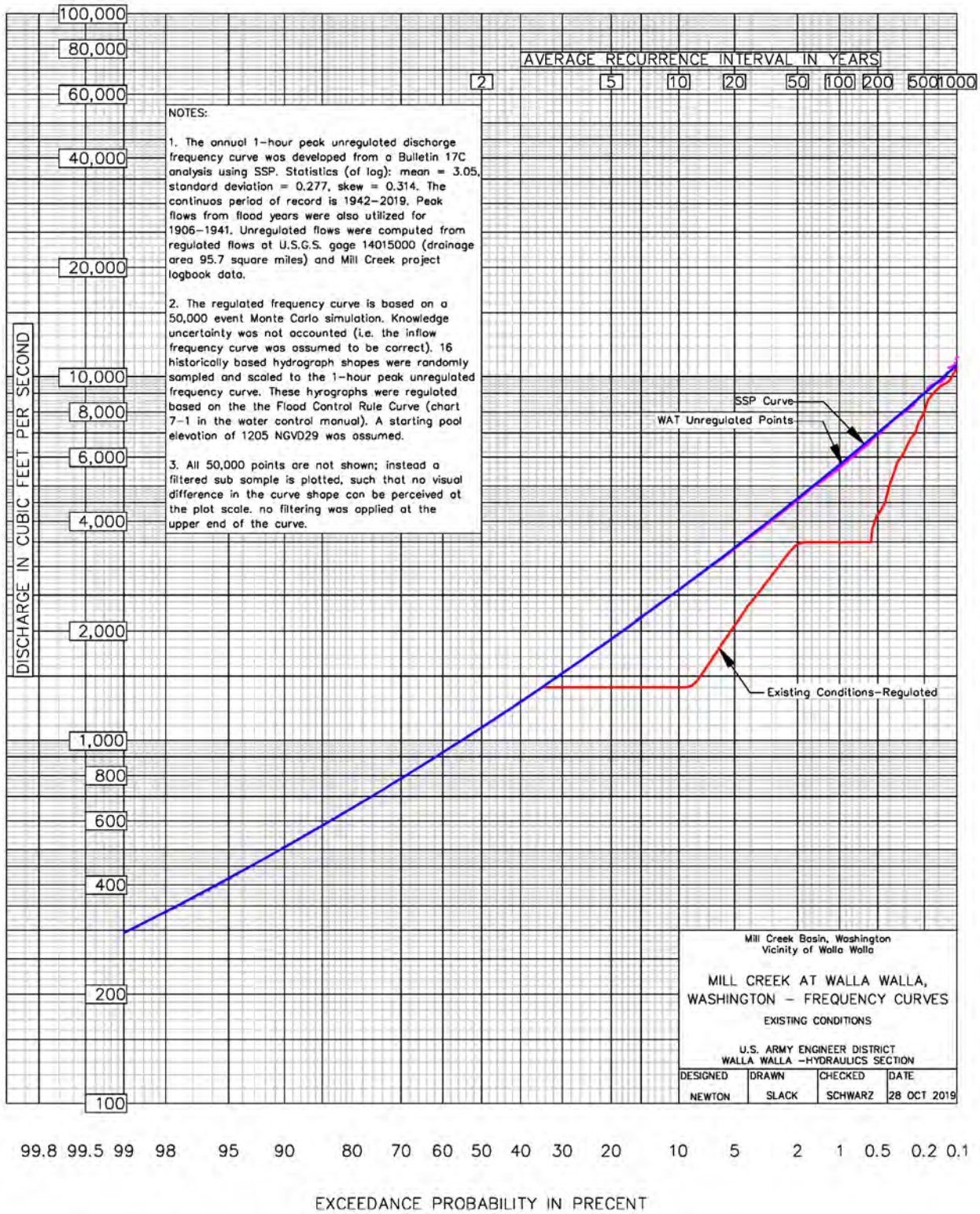

Figure 3-6. Existing Conditions Regulated Flow Frequency Curve

This operation maintains flows to the channel flow target of 3,500 cfs up to the 0.568-percent chance flood (175 Year).

The existing conditions regulated frequency curve was entered into HEC-FDA as a graphical frequency curve. The following data points were input into the HEC-FDA model with a period of record of 78 years.

Table 3-4. Existing Conditions Regulated Frequency Curve FDA Input

Annual Exceedance Probability	Discharge (cfs)
0.9952	261
0.9828	327
0.9536	406
0.9	510
0.6	927
0.5	1081
0.4104	1249
0.3471	1399
0.25	1400
0.0913	1401
0.05	2069
0.02194	3336
0.01	3500
0.00568	3501
0.00566	3702
0.0052	4107
0.00458	4365
0.00416	4697
0.00404	5005
0.00232	7171
0.002	7977
0.00184	8639
0.0014	9371
0.001	10604
0.00018	16960
0.0001	19670

From these points, HEC-FDA generated the following frequency curve data and plot.

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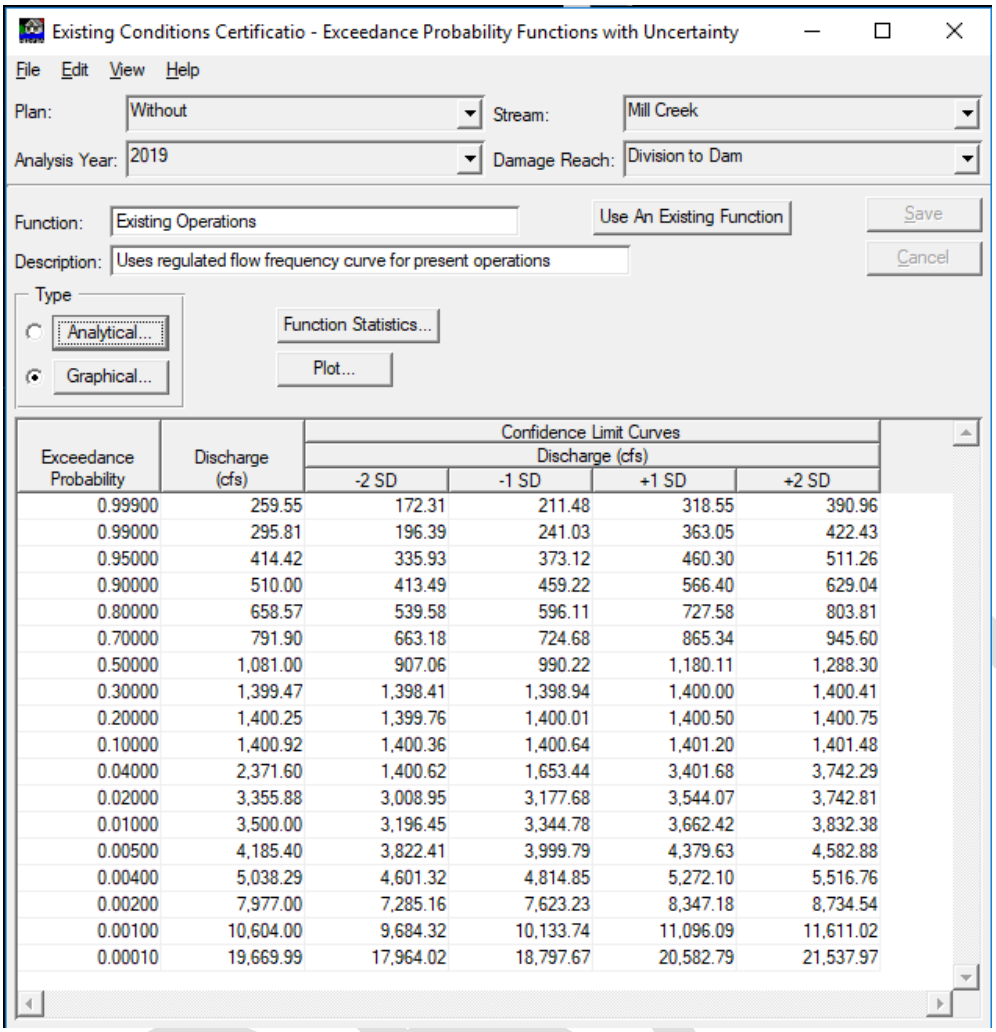

Exceedance Probability	Discharge (cfs)	Confidence Limit Curves Discharge (cfs) -2 SD	-1 SD	+1 SD	+2 SD
0.99900	259.55	172.31	211.48	318.55	390.96
0.99000	295.81	196.39	241.03	363.05	422.43
0.95000	414.42	335.93	373.12	460.30	511.26
0.90000	510.00	413.49	459.22	566.40	629.04
0.80000	658.57	539.58	596.11	727.58	803.81
0.70000	791.90	663.18	724.68	865.34	945.60
0.50000	1,081.00	907.06	990.22	1,180.11	1,288.30
0.30000	1,399.47	1,398.41	1,398.94	1,400.00	1,400.41
0.20000	1,400.25	1,399.76	1,400.01	1,400.50	1,400.75
0.10000	1,400.92	1,400.36	1,400.64	1,401.20	1,401.48
0.04000	2,371.60	1,400.62	1,653.44	3,401.68	3,742.29
0.02000	3,355.88	3,008.95	3,177.68	3,544.07	3,742.81
0.01000	3,500.00	3,196.45	3,344.78	3,662.42	3,832.38
0.00500	4,185.40	3,822.41	3,999.79	4,379.63	4,582.88
0.00400	5,038.29	4,601.32	4,814.85	5,272.10	5,516.76
0.00200	7,977.00	7,285.16	7,623.23	8,347.18	8,734.54
0.00100	10,604.00	9,684.32	10,133.74	11,096.09	11,611.02
0.00010	19,669.99	17,964.02	18,797.67	20,582.79	21,537.97

Figure 3-7. Existing Conditions Regulated Frequency Curve FDA Extrapolation

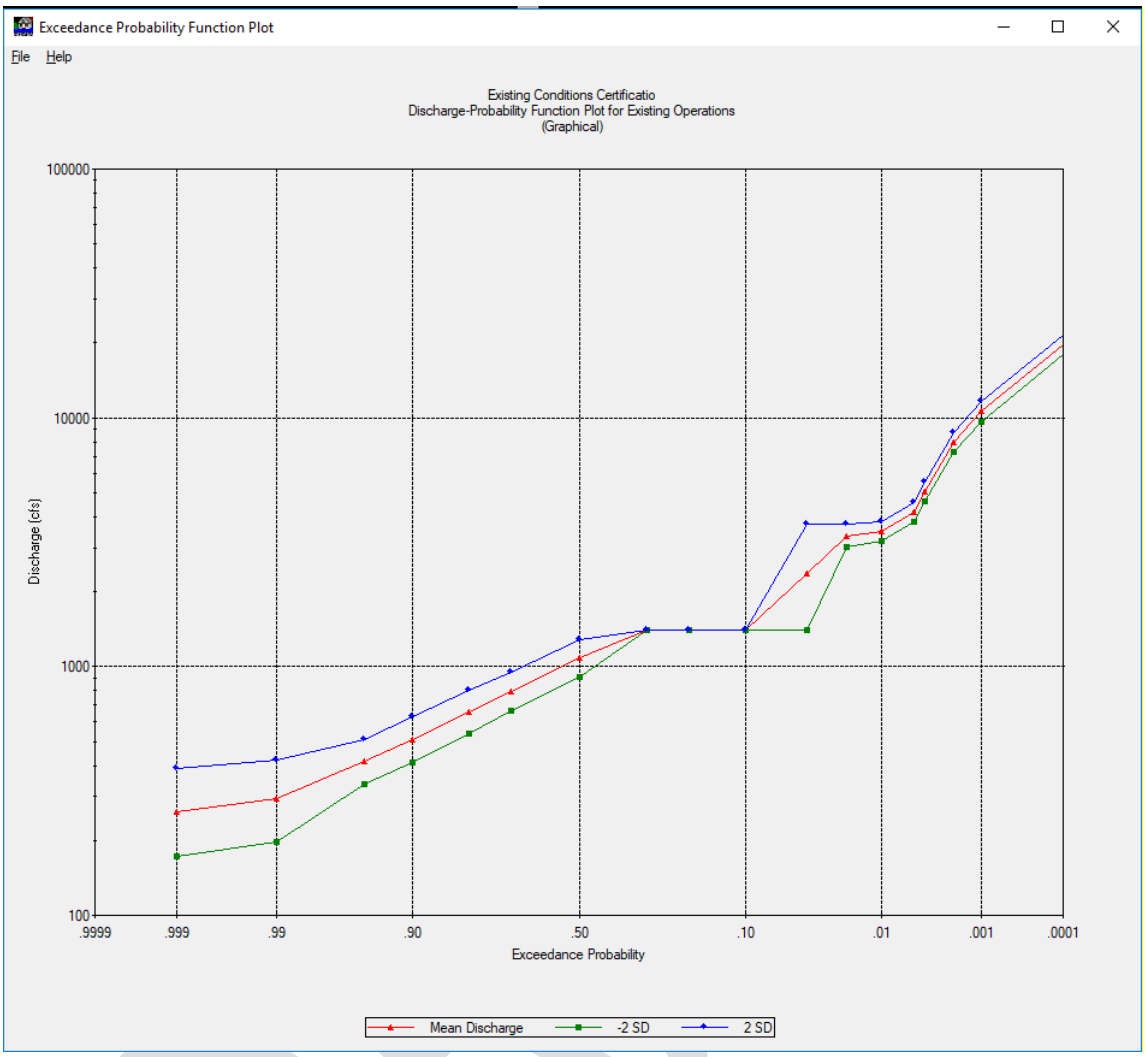

Figure 3-8. Existing Conditions Regulated Frequency Curve in FDA

Note that HEC-FDA extracts values from the entered data for predetermined selected frequencies. This hard fix in the model software diminishes the resolution and reduces the preciseness of the results.

HEC-FDA does not provide the expected uncertainty breadth in the frequency curve when there are areas of small increases in flows with diminishing probabilities, when using the graphical frequency curve approach. This is the reason that the frequency curve confidence bounds narrow particularly for the 1,400 cfs regulation target and to a lesser degree for the 3,500 cfs regulation target. We also do not see the expected spread in uncertainty as you move from the 1% flood to the 0.1% flood. These are limitations in HEC-FDA software that need to be understood in the context of this study. HEC was consulted on these issues and an alternative method will be used for the final model runs that inputs the unregulated frequency curve and then uses a transform function to regulate. This should better define the confidence limits.

Small Excavation

The regulated flow frequency curve from HEC-WAT output is shown in Figure 3-9.

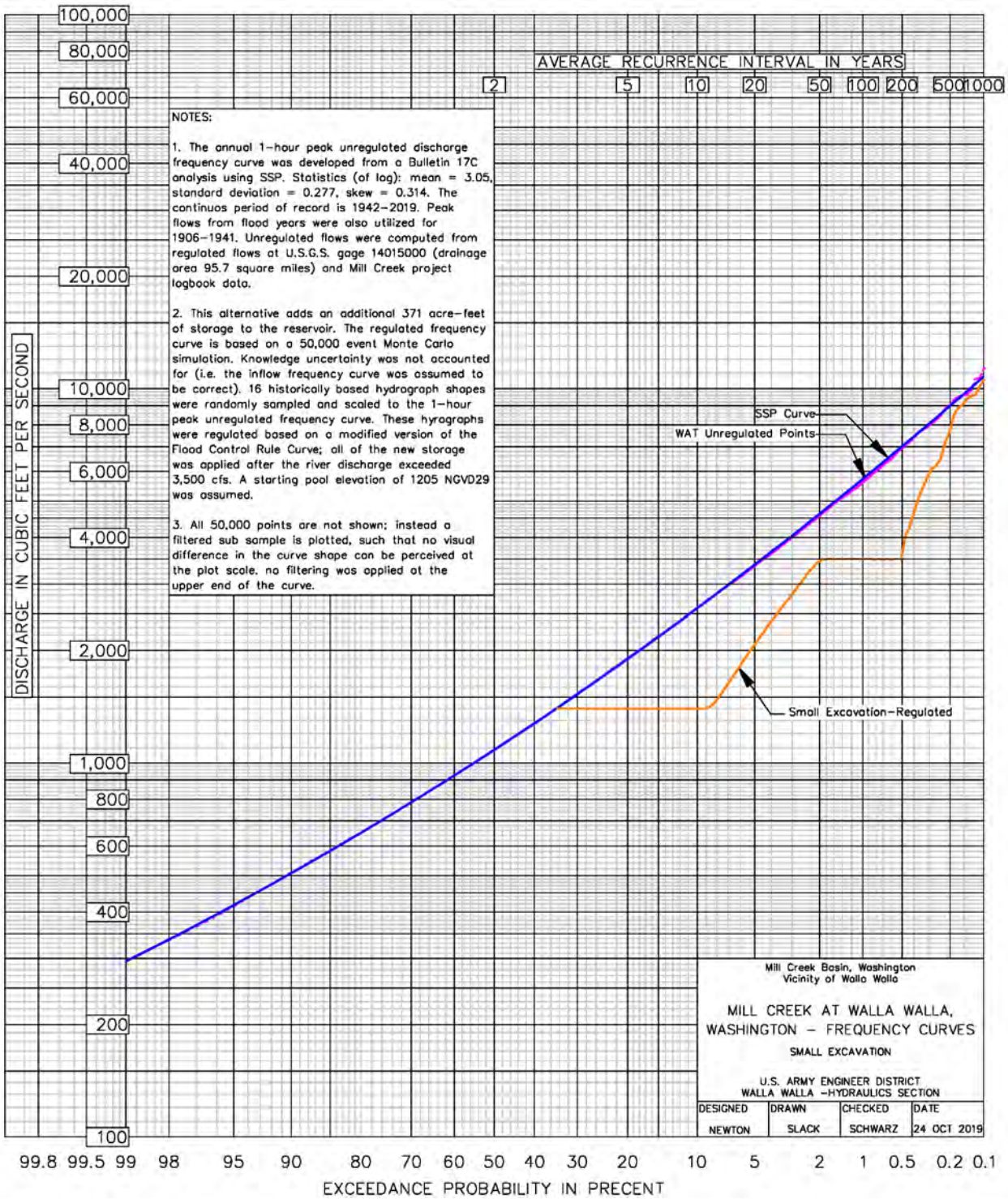

Figure 3-9. Small Excavation Regulated Flow Frequency Curve

This operation maintains flows to the channel flow target of 3,500 cfs up to the 0.506-percent chance flood (196 Year).

The small excavation regulated frequency curve was entered into HEC-FDA as a graphical frequency curve. The following data points were input into the HEC-FDA model with a period of record of 78 years.

Table 3-5. Small Excavation Regulated Frequency Curve FDA Input

Annual Exceedance Probability	Discharge (cfs)
0.9952	261
0.9828	327
0.9536	406
0.9	510
0.6	927
0.5	1081
0.4104	1249
0.3471	1399
0.25	1400
0.0913	1401
0.05	2069
0.02194	3336
0.01	3500
0.00506	3501
0.00226	6803
0.00186	8476
0.00144	9371
0.001	10604
0.00018	16960
0.0001	19670

From these points, HEC-FDA generated the following frequency curve data and plot.

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File Edit View Help

Plan: Without | Stream: Mill Creek

Analysis Year: 2019 | Damage Reach: Division to Dam

Function: Small Excavation | Use An Existing Function | Save

Description: Uses regulated flow frequency curve | Cancel

Type: Analytical... / Graphical... | Function Statistics... | Plot...

Exceedance Probability	Discharge (cfs)	Confidence Limit Curves Discharge (cfs) -2 SD	-1 SD	+1 SD	+2 SD
0.99900	259.55	172.31	211.48	318.55	390.96
0.99000	295.81	196.39	241.03	363.05	422.43
0.95000	414.42	335.93	373.12	460.30	511.26
0.90000	510.00	413.49	459.22	566.40	629.04
0.80000	658.57	539.58	596.11	727.58	803.81
0.70000	791.90	663.18	724.68	865.34	945.60
0.50000	1,081.00	907.06	990.22	1,180.11	1,288.30
0.30000	1,386.61	1,360.32	1,373.40	1,399.94	1,400.41
0.20000	1,400.25	1,399.76	1,400.01	1,400.50	1,400.75
0.10000	1,400.92	1,400.36	1,400.64	1,401.20	1,401.48
0.04000	2,371.60	1,400.62	1,653.44	3,401.68	3,742.29
0.02000	3,355.88	3,008.95	3,177.68	3,544.07	3,742.81
0.01000	3,500.00	3,196.56	3,344.84	3,662.36	3,832.25
0.00500	3,532.92	3,226.63	3,376.30	3,696.80	3,868.29
0.00400	4,177.91	3,815.70	3,992.70	4,371.72	4,574.51
0.00200	7,597.28	6,938.61	7,260.48	7,949.70	8,318.47
0.00100	10,604.00	9,684.66	10,133.92	11,095.89	11,610.61
0.00010	19,669.99	17,964.65	18,798.00	20,582.43	21,537.21

Figure 3-10. Small Excavation Regulated Frequency Curve FDA Extrapolation

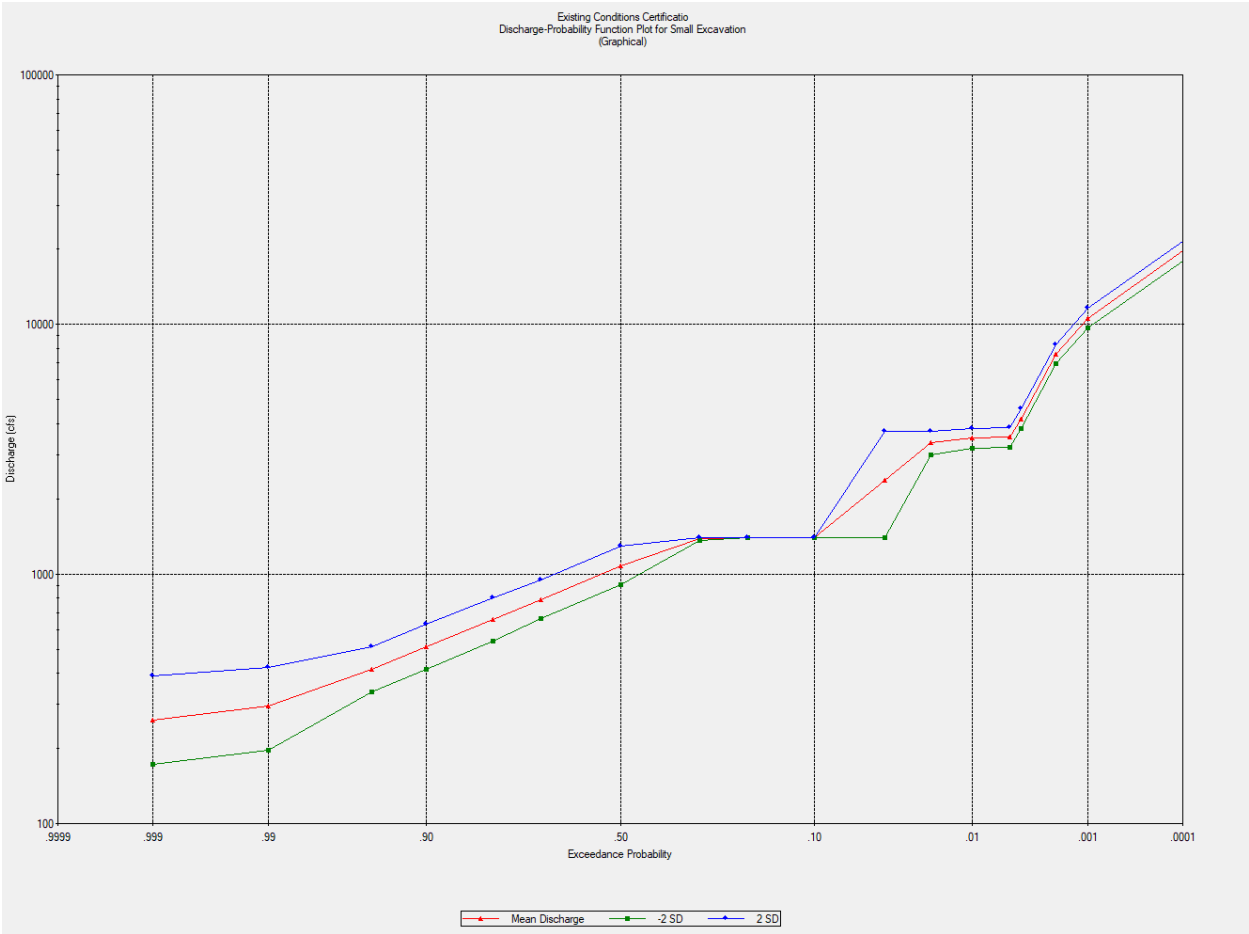

Figure 3-11. Small Excavation Regulated Frequency Curve in FDA

Note that HEC-FDA extracts values from the entered data for predetermined selected frequencies. This hard fix in the model software diminishes the resolution and reduces the preciseness of the results.

HEC-FDA does not provide the expected uncertainty breadth in the frequency curve when there are areas of small increases in flows with diminishing probabilities, when using the graphical frequency curve approach. We do not see the expected spread in uncertainty as you move from the 1% flood to the 0.1% flood. These are limitations in HEC-FDA software that need to be understood in the context of this study. HEC was consulted on these issues and an alternative method will be used for the final model runs that inputs the unregulated frequency curve and then uses a transform function to regulate. This should better define the confidence limits.

Moderate Excavation

The regulated flow frequency curve from HEC-WAT output is shown in the following Figure.

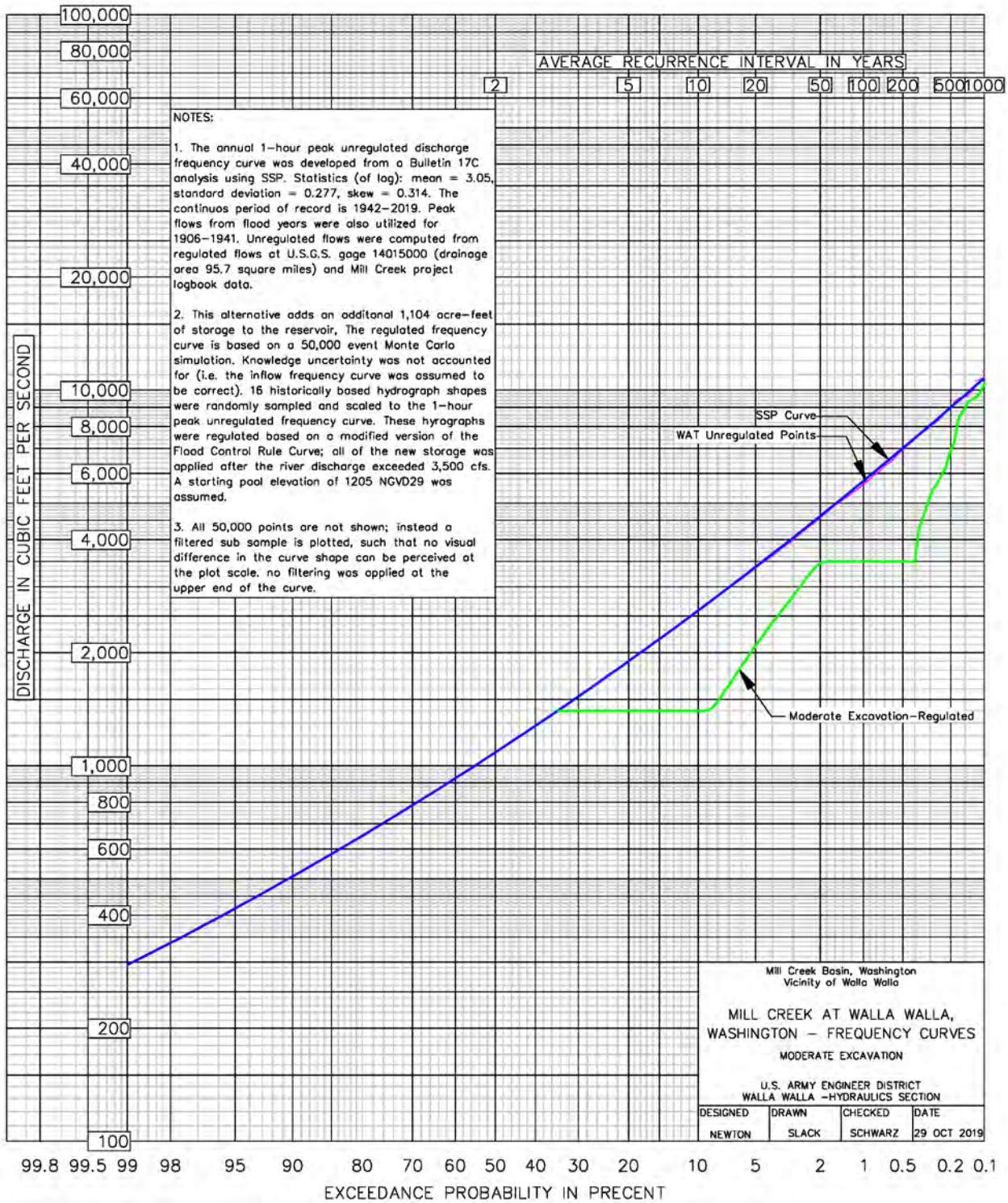

Figure 3-12. Moderate Excavation Regulated Flow Frequency Curve

This operation maintains flows to the channel flow target of 3,500 cfs up to the 0.398-percent chance flood (250 Year).

The moderate excavation regulated frequency curve was entered into HEC-FDA as a graphical frequency curve. The following data points were input into the HEC-FDA model with a period of record of 78 years.

Table 3-6. Moderate Excavation Regulated Frequency Curve FDA Input

Annual Exceedance Probability	Discharge (cfs)
0.9952	261
0.9828	327
0.9536	406
0.9	510
0.6	927
0.5	1081
0.4104	1249
0.3471	1399
0.25	1400
0.0913	1401
0.05	2069
0.02194	3336
0.01	3500
0.00398	3501
0.00382	4164
0.00222	6162
0.00142	9264
0.001	10604
0.00018	16960
0.0001	19670

From these points, FDA generated the following frequency curve data and plot.

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File Edit View Help

Plan: Without | Stream: Mill Creek

Analysis Year: 2019 | Damage Reach: Division to Dam

Function: Moderate Excavation | Use An Existing Function | Save

Description: Uses regulated flow frequency curve | Cancel

Type: Analytical... / Graphical...

Function Statistics... | Plot...

Exceedance Probability	Discharge (cfs)	Confidence Limit Curves Discharge (cfs) -2 SD	-1 SD	+1 SD	+2 SD
0.99900	259.55	172.31	211.48	318.55	390.96
0.99000	295.81	196.39	241.03	363.05	422.43
0.95000	414.42	335.93	373.12	460.30	511.26
0.90000	510.00	413.49	459.22	566.40	629.04
0.80000	658.57	539.58	596.11	727.58	803.81
0.70000	791.90	663.18	724.68	865.34	945.60
0.50000	1,081.00	907.06	990.22	1,180.11	1,288.30
0.30000	1,386.61	1,360.32	1,373.40	1,399.94	1,400.41
0.20000	1,400.25	1,399.76	1,400.01	1,400.50	1,400.75
0.10000	1,400.92	1,400.36	1,400.64	1,401.20	1,401.48
0.04000	2,371.60	1,400.62	1,653.44	3,401.68	3,742.29
0.02000	3,355.88	3,008.95	3,177.68	3,544.07	3,742.81
0.01000	3,500.00	3,196.70	3,344.91	3,662.28	3,832.08
0.00500	3,500.76	3,197.39	3,345.64	3,663.08	3,832.91
0.00400	3,500.99	3,197.61	3,345.86	3,663.32	3,833.17
0.00200	6,787.70	6,199.49	6,486.93	7,102.41	7,431.71
0.00100	10,604.00	9,685.08	10,134.13	11,095.66	11,610.11
0.00010	19,669.99	17,965.42	18,798.40	20,582.00	21,536.29

Figure 3-13. Moderate Excavation Regulated Frequency Curve FDA Extrapolation

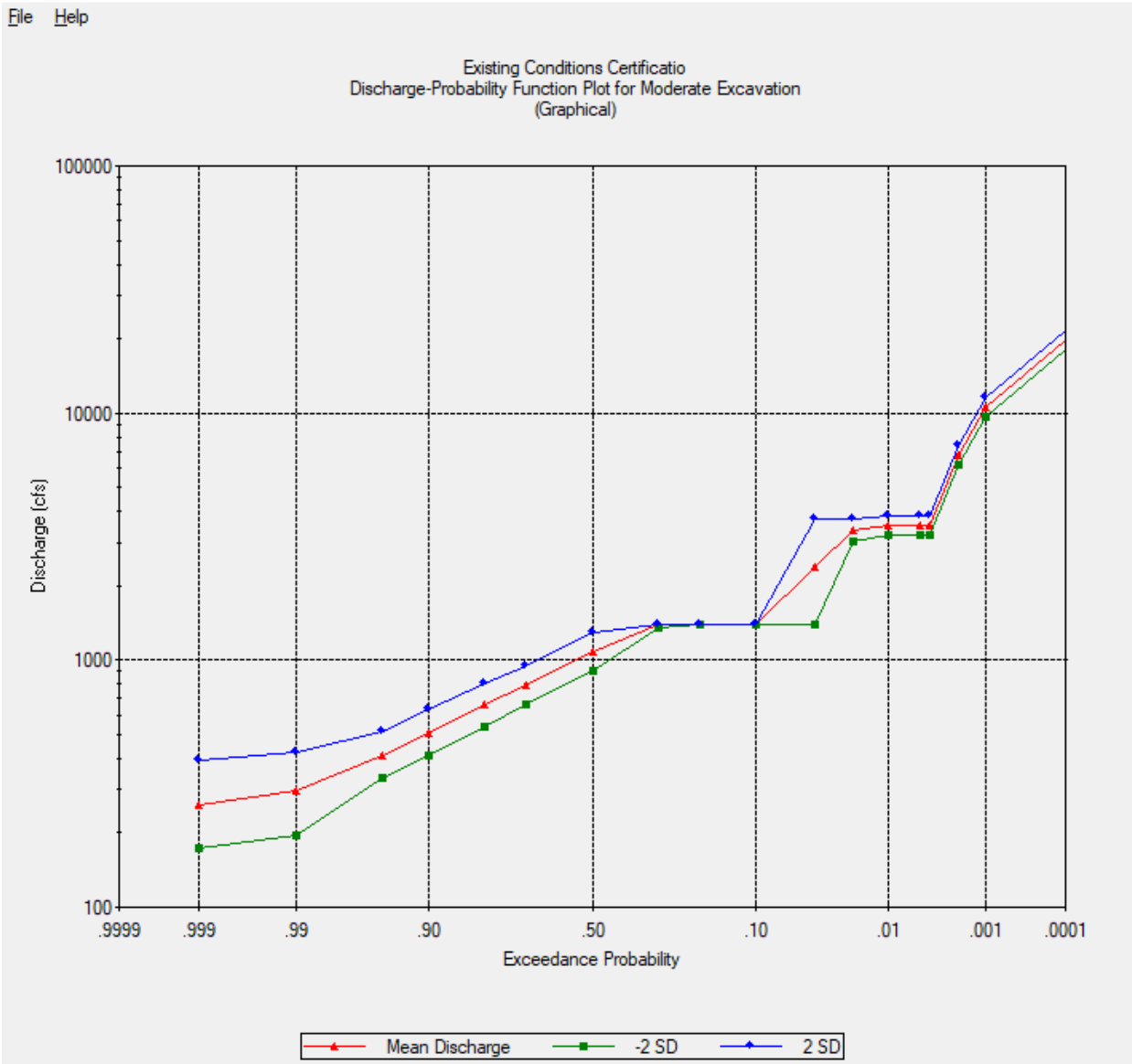


Figure 3-14. Moderate Excavation Regulated Frequency Curve in FDA

Note that HEC-FDA extracts values from the entered data for predetermined selected frequencies. This hard fix in the model software diminishes the resolution and reduces the preciseness of the results.

HEC-FDA does not provide the expected uncertainty breadth in the frequency curve when there are areas of small increases in flows with diminishing probabilities, when using the graphical frequency curve approach. We do not see the expected spread in uncertainty as you move from the 1% flood to the 0.1% flood. These are limitations in HEC-FDA software that need to be understood in the context of this study. HEC was consulted on these issues and an alternative method will be used for the final model runs that inputs the unregulated frequency curve and then uses a transform function to regulate. This should better define the confidence limits.

Small Levee Raise

The regulated flow frequency curve from HEC-WAT output is shown in Figure 3-15.

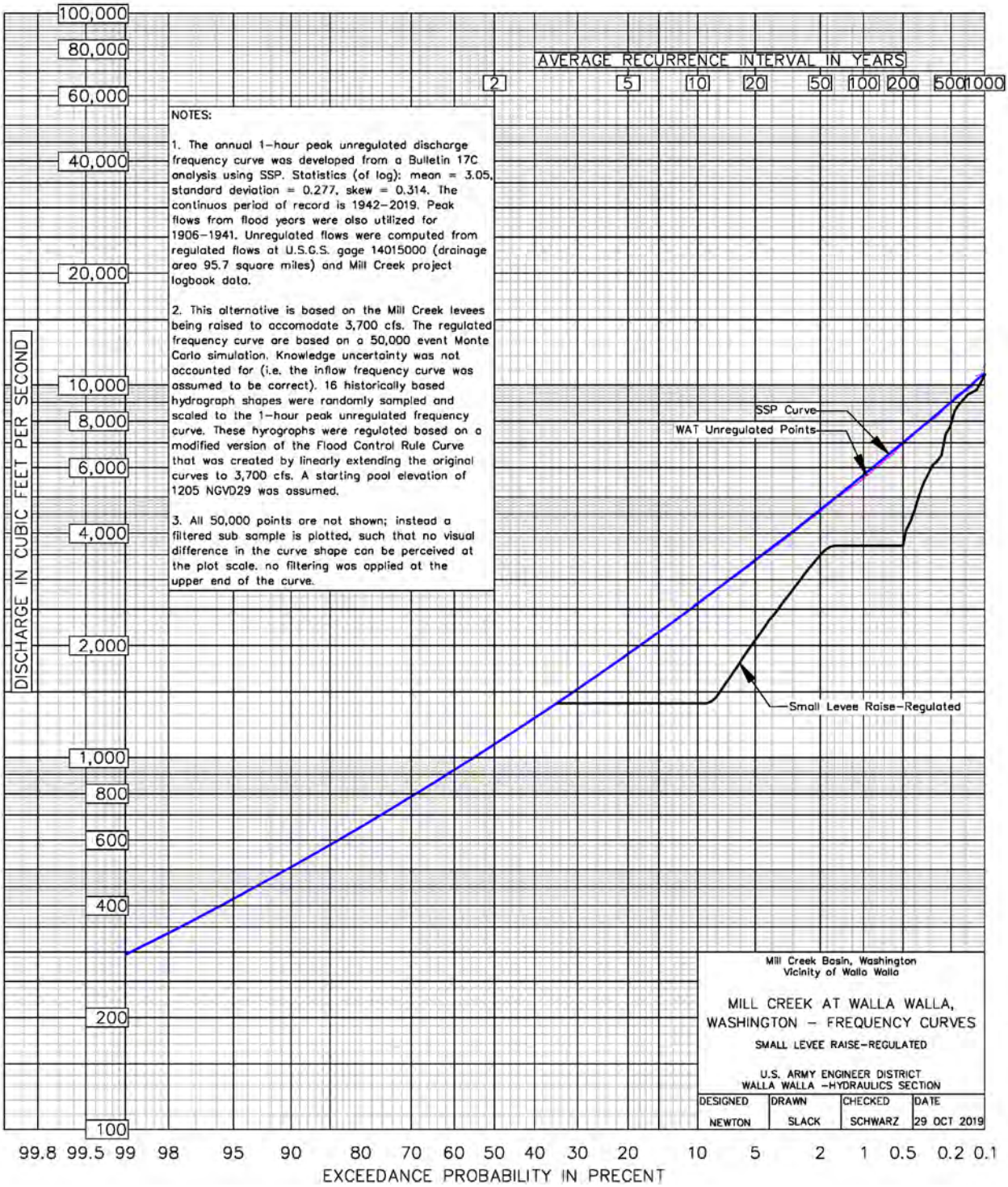


Figure 3-15. Small Levee Raise Regulated Flow Frequency Curve

This operation maintains flows to the channel flow target of 3,700 cfs up to the 0.5-percent chance flood (200 Year).

The small levee raise regulated frequency curve was entered into HEC-FDA as a graphical frequency curve. The following data points were input into the HEC-FDA model with a period of record of 78 years.

Table 3-7. Small Levee Raise Regulated Frequency Curve FDA Input

Annual Exceedance Probability	Discharge (cfs)
0.9952	261
0.9828	327
0.9536	406
0.9	510
0.6	927
0.5	1081
0.4104	1249
0.3471	1399
0.25	1400
0.0913	1401
0.05	2069
0.02194	3336
0.01	3700
0.005	3701
0.004	4622
0.00334	5634
0.00236	6494
0.00226	7272
0.0017	8875
0.00144	9371
0.001	10604
0.00018	16960
0.0001	19670

From these points, HEC-FDA generated the following frequency curve data and plot.

DRAFT

Raise 3700 Certification - Exceedance Probability Functions with Uncertainty

File Edit View Help

Plan: Without | Stream: Mill Creek

Analysis Year: 2019 | Damage Reach: Division to Dam

Function: 3700 cfs levee raise | Use An Existing Function | Save

Description: regulated frequency curve for operations assuming a levee raise | Cancel

Type: Analytical... / Graphical... | Function Statistics... | Plot...

Exceedance Probability	Discharge (cfs)	Confidence Limit Curves Discharge (cfs) -2 SD	-1 SD	+1 SD	+2 SD
0.99900	259.55	172.31	211.48	318.55	390.96
0.99000	295.81	196.39	241.03	363.05	422.43
0.95000	414.42	335.93	373.12	460.30	511.26
0.90000	510.00	413.49	459.22	566.40	629.04
0.80000	658.57	539.58	596.11	727.58	803.81
0.70000	791.90	663.18	724.68	865.34	945.60
0.50000	1,081.00	907.06	990.22	1,180.11	1,288.30
0.30000	1,399.47	1,398.41	1,398.94	1,400.00	1,400.41
0.20000	1,400.25	1,399.76	1,400.01	1,400.50	1,400.75
0.10000	1,400.92	1,400.36	1,400.64	1,401.20	1,401.48
0.04000	2,371.60	1,400.62	1,653.44	3,401.68	4,275.64
0.02000	3,379.04	2,670.09	3,003.72	3,801.26	4,276.23
0.01000	3,700.00	3,043.00	3,355.46	4,079.92	4,498.84
0.00500	3,701.00	3,043.83	3,356.37	4,081.02	4,500.06
0.00400	4,622.00	3,801.29	4,191.60	5,096.59	5,619.91
0.00200	7,926.22	6,518.79	7,188.14	8,740.09	9,637.52
0.00100	10,604.00	8,721.09	9,616.57	11,692.82	12,893.45
0.00010	19,669.99	16,177.25	17,838.35	21,689.71	23,916.83

Figure 3-16. Small Levee Raise Regulated Frequency Curve FDA Extrapolation

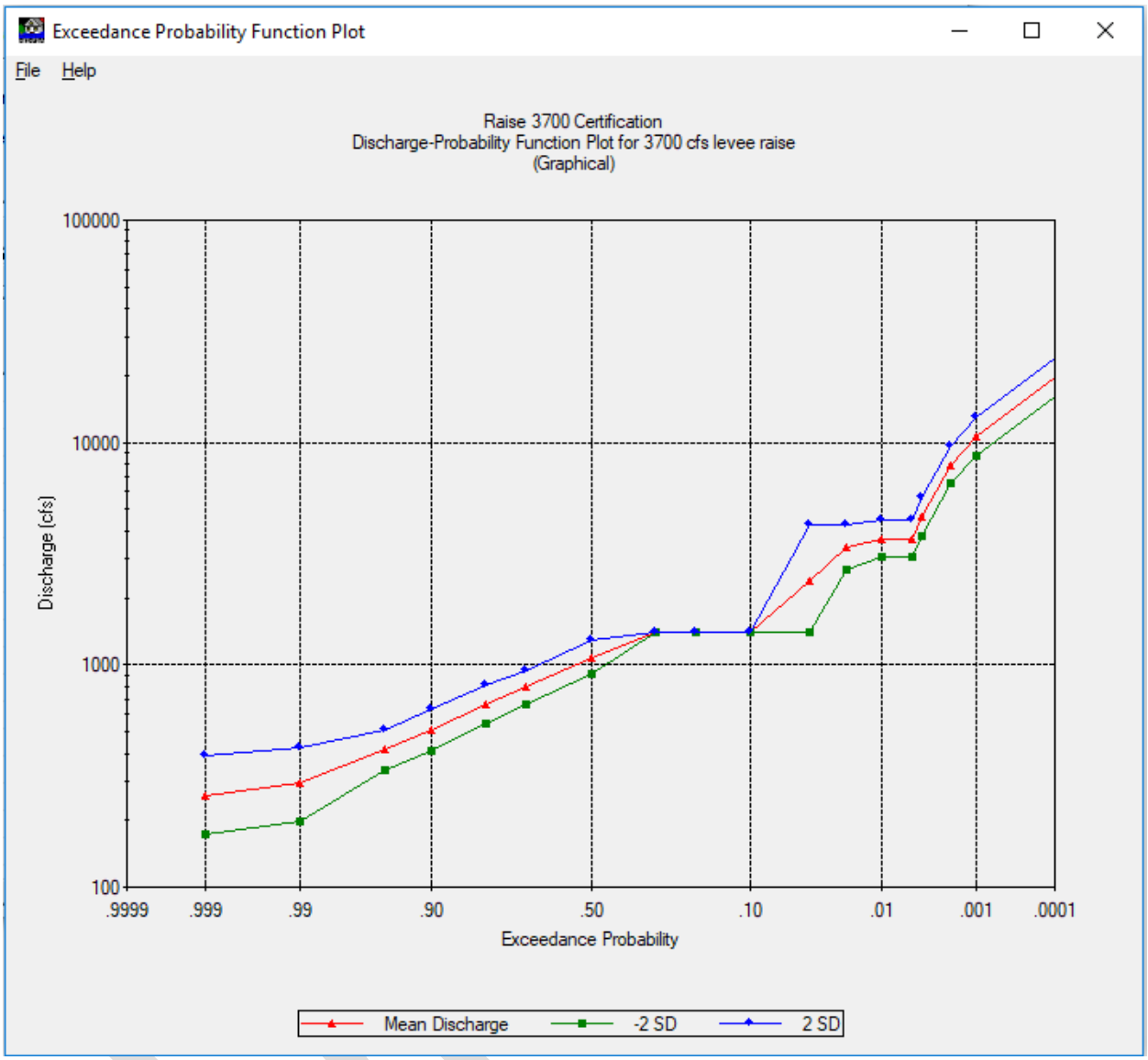


Figure 3-17. Small Levee Raise Regulated Frequency Curve in FDA

Note that HEC-FDA extracts values from the entered data for predetermined selected frequencies. This hard fix in the model software diminishes the resolution and reduces the preciseness of the results.

HEC-FDA does not provide the expected uncertainty breadth in the frequency curve when there are areas of small increases in flows with diminishing probabilities, when using the graphical frequency curve approach. We do not see the expected spread in uncertainty as you move from the 1% flood to the 0.1% flood. These are limitations in HEC-FDA software that need to be understood in the context of this study. HEC was consulted on these issues and an alternative method will be used for the final model runs that inputs the unregulated frequency curve and then uses a transform function to regulate. This should better define the confidence limits.

Moderate Levee Raise

The regulated flow frequency curve from HEC-WAT output is shown in Figure 3-18.

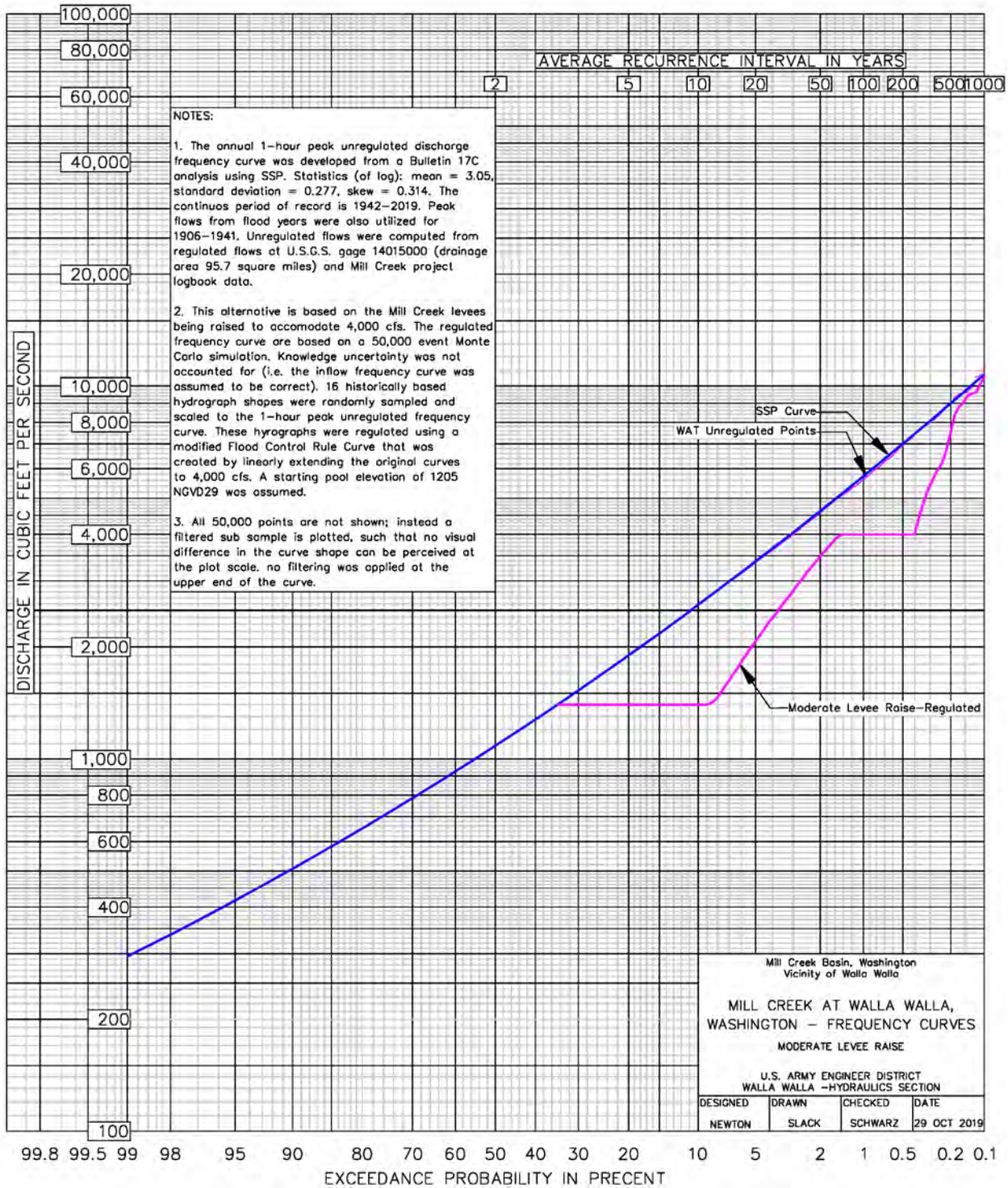


Figure 3-18. Moderate Levee Raise Regulated Flow Frequency Curve

This operation maintains flows to the channel flow target of 4,000 cfs up to the 0.4-percent chance flood (250 Year).

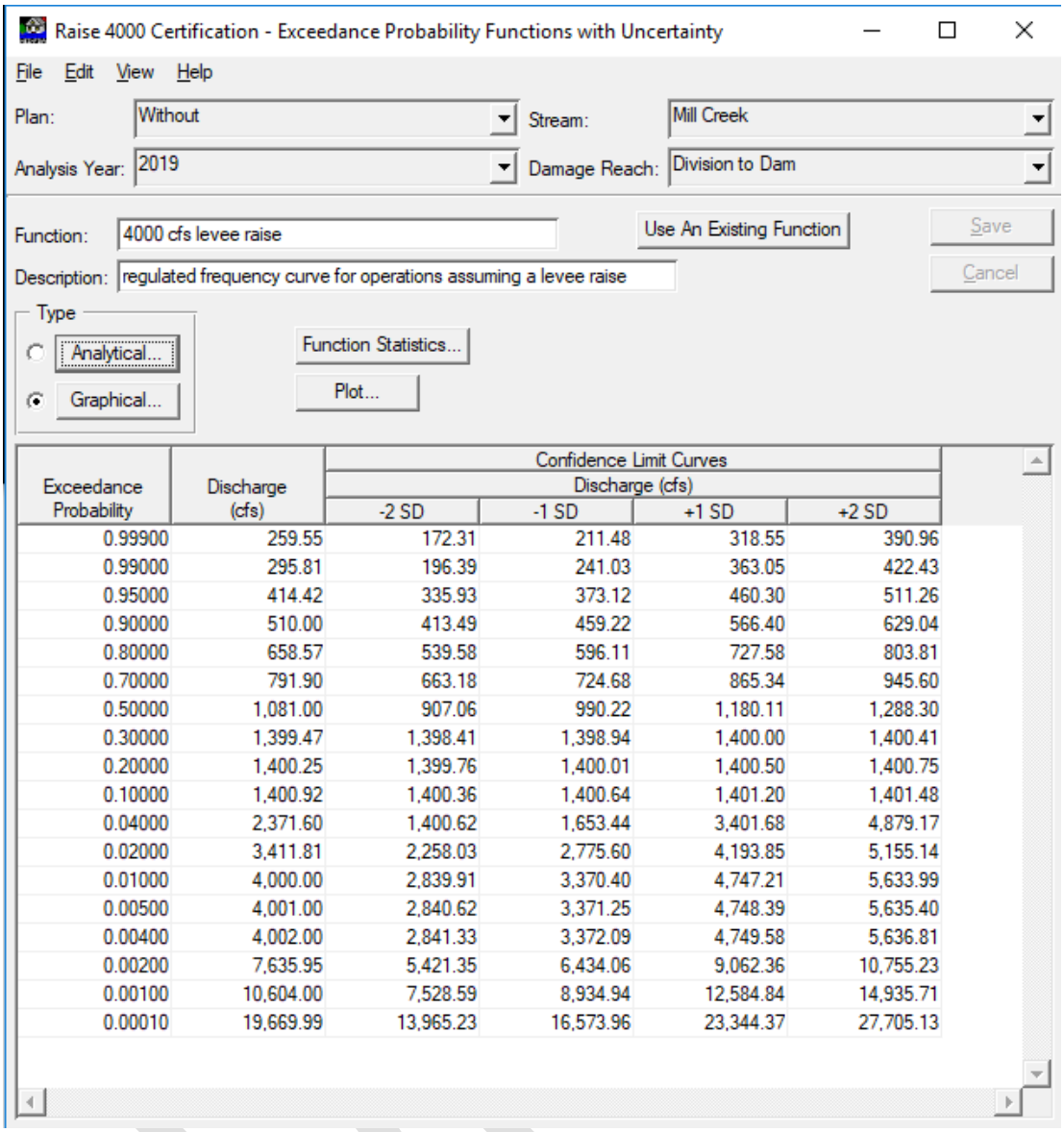
The regulated frequency curve was entered into HEC-FDA as a graphical frequency curve. The following data points were input into the HEC-FDA model with a period of record of 78 years.

Table 3-8. Moderate Levee Raise Regulated Frequency Curve FDA Input

Annual Exceedance Probability	Discharge (cfs)
0.9952	261
0.9828	327
0.9536	406
0.9	510
0.6	927
0.5	1081
0.4104	1249
0.3471	1399
0.25	1400
0.0913	1401
0.05	2069
0.02194	3336
0.01	4000
0.005	4001
0.004	4002
0.0022	6517
0.0019	8311
0.00144	9371
0.001	10604
0.00018	16960
0.0001	19670

From these points, FDA generated the following frequency curve data and plot.

DRAFT



Exceedance Probability	Discharge (cfs)	Confidence Limit Curves Discharge (cfs) -2 SD	-1 SD	+1 SD	+2 SD
0.99900	259.55	172.31	211.48	318.55	390.96
0.99000	295.81	196.39	241.03	363.05	422.43
0.95000	414.42	335.93	373.12	460.30	511.26
0.90000	510.00	413.49	459.22	566.40	629.04
0.80000	658.57	539.58	596.11	727.58	803.81
0.70000	791.90	663.18	724.68	865.34	945.60
0.50000	1,081.00	907.06	990.22	1,180.11	1,288.30
0.30000	1,399.47	1,398.41	1,398.94	1,400.00	1,400.41
0.20000	1,400.25	1,399.76	1,400.01	1,400.50	1,400.75
0.10000	1,400.92	1,400.36	1,400.64	1,401.20	1,401.48
0.04000	2,371.60	1,400.62	1,653.44	3,401.68	4,879.17
0.02000	3,411.81	2,258.03	2,775.60	4,193.85	5,155.14
0.01000	4,000.00	2,839.91	3,370.40	4,747.21	5,633.99
0.00500	4,001.00	2,840.62	3,371.25	4,748.39	5,635.40
0.00400	4,002.00	2,841.33	3,372.09	4,749.58	5,636.81
0.00200	7,635.95	5,421.35	6,434.06	9,062.36	10,755.23
0.00100	10,604.00	7,528.59	8,934.94	12,584.84	14,935.71
0.00010	19,669.99	13,965.23	16,573.96	23,344.37	27,705.13

Figure 3-19. Moderate Levee Raise Regulated Frequency Curve FDA Extrapolation

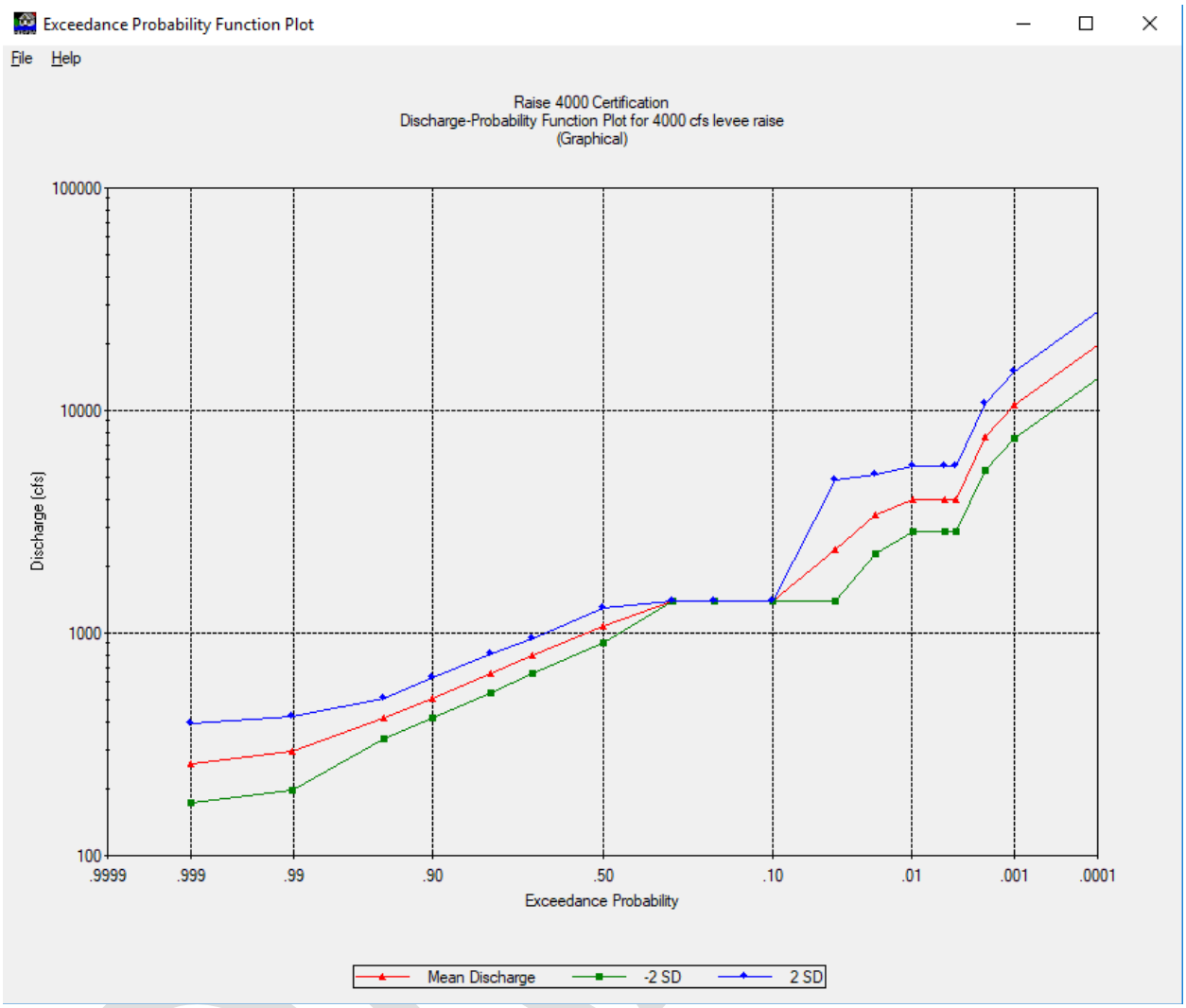


Figure 3-20. Moderate Levee Raise Regulated Frequency Curve in FDA

Note that HEC-FDA extracts values from the entered data for predetermined selected frequencies. This hard fix in the model software diminishes the resolution and reduces the preciseness of the results.

HEC-FDA does not provide the expected uncertainty breadth in the frequency curve when there are areas of small increases in flows with diminishing probabilities, when using the graphical frequency curve approach. We do not see the expected spread in uncertainty as you move from the 1% flood to the 0.1% flood. These are limitations in HEC-FDA software that need to be understood in the context of this study. HEC was consulted on these issues and an alternative method will be used for the final model runs that inputs the unregulated frequency curve and then uses a transform function to regulate. This should better define the confidence limits.

Small Excavation and Small Levee Raise

The regulated flow frequency curve from HEC-WAT output is shown in Figure 3-21.

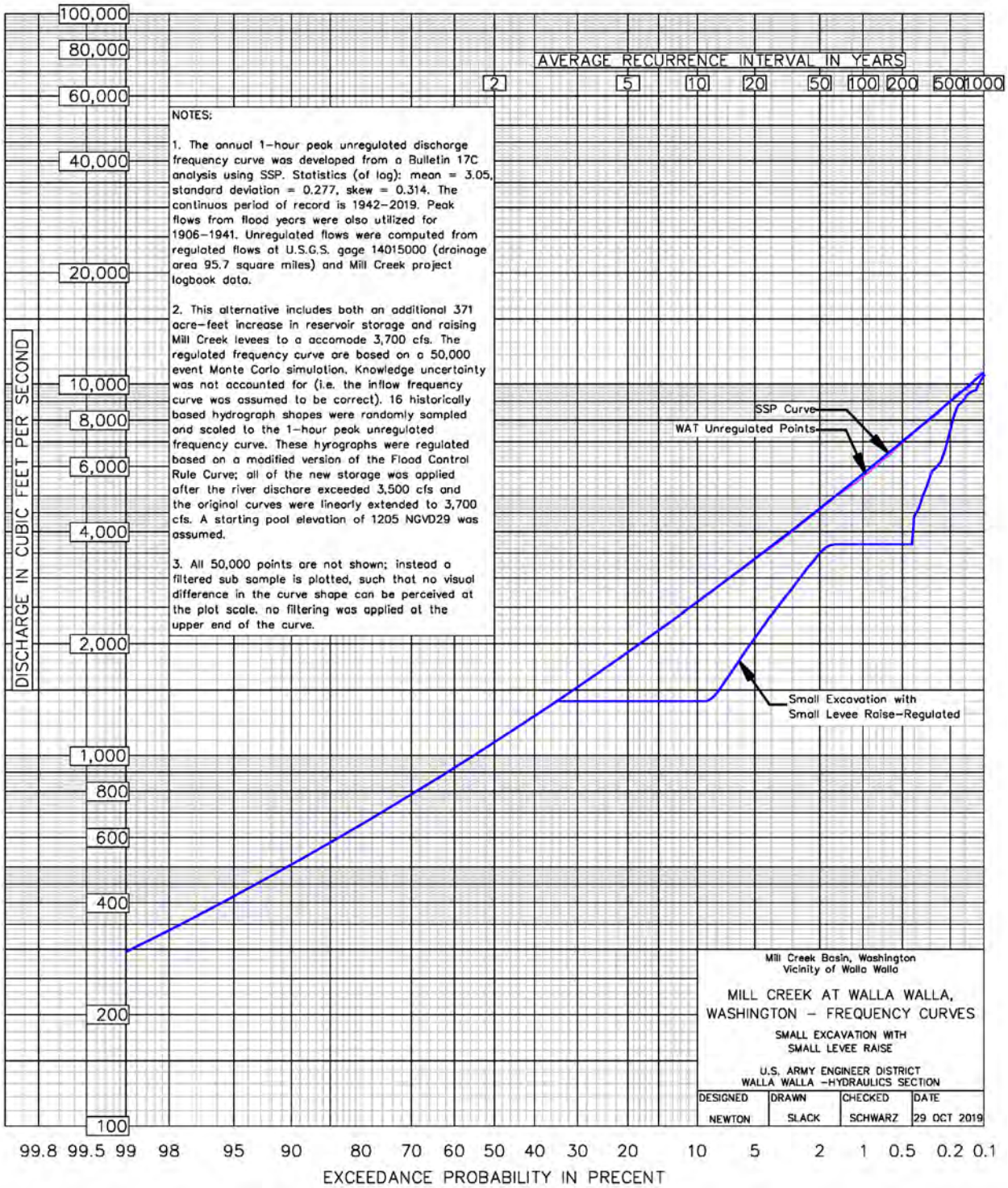


Figure 3-21. Small Excavation/Small Levee Raise Regulated Flow Frequency Curve

This operation maintains flows to the channel flow target of 3,700 cfs up to the 0.418-percent chance flood (238 Year).

The regulated frequency curve was entered into HEC-FDA as a graphical frequency curve. The following data points were input into the HEC-FDA model with a period of record of 78 years.

Table 3-9. Small Excavation and Small Levee Raise Regulated Frequency Curve FDA Input

Annual Exceedance Probability	Discharge (cfs)
0.9952	261
0.9828	327
0.9536	406
0.9	510
0.6	927
0.5	1081
0.4104	1249
0.3471	1399
0.25	1400
0.0913	1401
0.05	2069
0.02194	3336
0.01	3700
0.00418	3701
0.0041	4225
0.00228	6432
0.00172	8778
0.00144	9264
0.001	10604
0.00018	16960
0.0001	19670

From these points, HEC-FDA generated the following frequency curve data and plot.

DRAFT

Function: Small Excavation and Small Levee

Use An Existing Function

Save

Description: Uses regulated flow frequency curve

Cancel

Type

Analytical...

Graphical...

Function Statistics...

Plot...

Exceedance Probability	Discharge (cfs)	Confidence Limit Curves Discharge (cfs) -2 SD	-1 SD	+1 SD	+2 SD
0.99900	259.55	172.31	211.48	318.55	390.96
0.99000	295.81	196.39	241.03	363.05	422.43
0.95000	414.42	335.93	373.12	460.30	511.26
0.90000	510.00	413.49	459.22	566.40	629.04
0.80000	658.57	539.58	596.11	727.58	803.81
0.70000	791.90	663.18	724.68	865.34	945.60
0.50000	1,081.00	907.06	990.22	1,180.11	1,288.30
0.30000	1,386.61	1,360.32	1,373.40	1,399.94	1,400.41
0.20000	1,400.25	1,399.76	1,400.01	1,400.50	1,400.75
0.10000	1,400.92	1,400.36	1,400.64	1,401.20	1,401.48
0.04000	2,371.60	1,400.62	1,653.44	3,401.68	4,275.64
0.02000	3,379.04	2,670.09	3,003.72	3,801.26	4,276.23
0.01000	3,700.00	3,043.10	3,355.51	4,079.85	4,498.71
0.00500	3,700.80	3,043.76	3,356.24	4,080.74	4,499.68
0.00400	4,302.55	3,538.67	3,901.96	4,744.26	5,231.33
0.00200	7,439.85	6,118.97	6,747.17	8,203.65	9,045.87
0.00100	10,604.00	8,721.35	9,616.72	11,692.64	12,893.04
0.00010	19,669.99	16,177.77	17,838.63	21,689.37	23,916.07

Figure 3-22. Small Excavation/Small Levee Raise Regulated Frequency Curve FDA Extrapolation

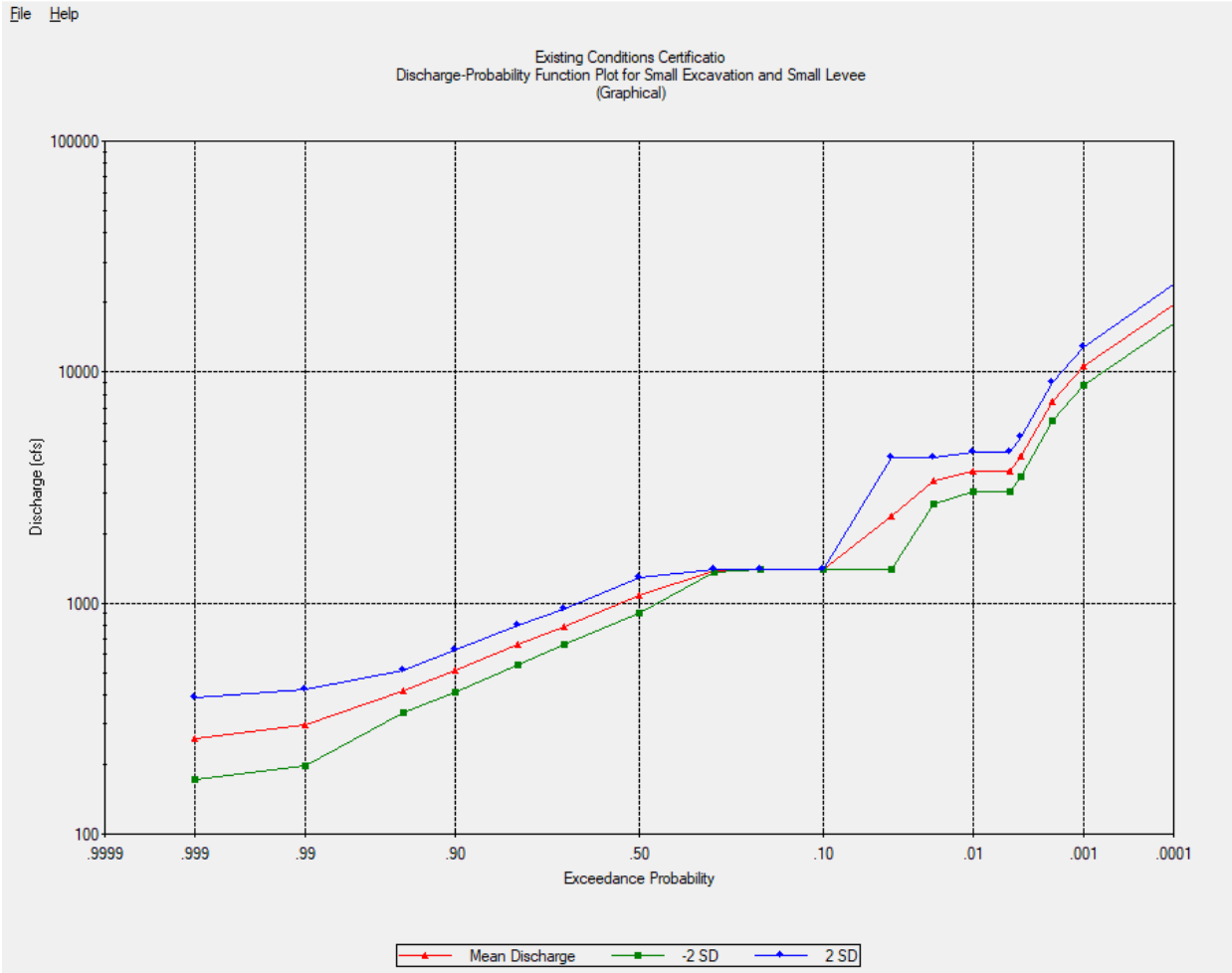


Figure 3-23. Small Excavation and Small Levee Raise Regulated Frequency Curve in FDA

Note that HEC-FDA extracts values from the entered data for predetermined selected frequencies. This hard fix in the model software diminishes the resolution and reduces the preciseness of the results.

HEC-FDA does not provide the expected uncertainty breadth in the frequency curve when there are areas of small increases in flows with diminishing probabilities, when using the graphical frequency curve approach. We do not see the expected spread in uncertainty as you move from the 1% flood to the 0.1% flood. These are limitations in HEC-FDA software that need to be understood in the context of this study. HEC was consulted on these issues and an alternative method will be used for the final model runs that inputs the unregulated frequency curve and then uses a transform function to regulate. This should better define the confidence limits.

Operational Change – Increased Initial Regulated Flow

The regulated flow frequency curve from HEC-WAT output is shown in Figure 3-24.

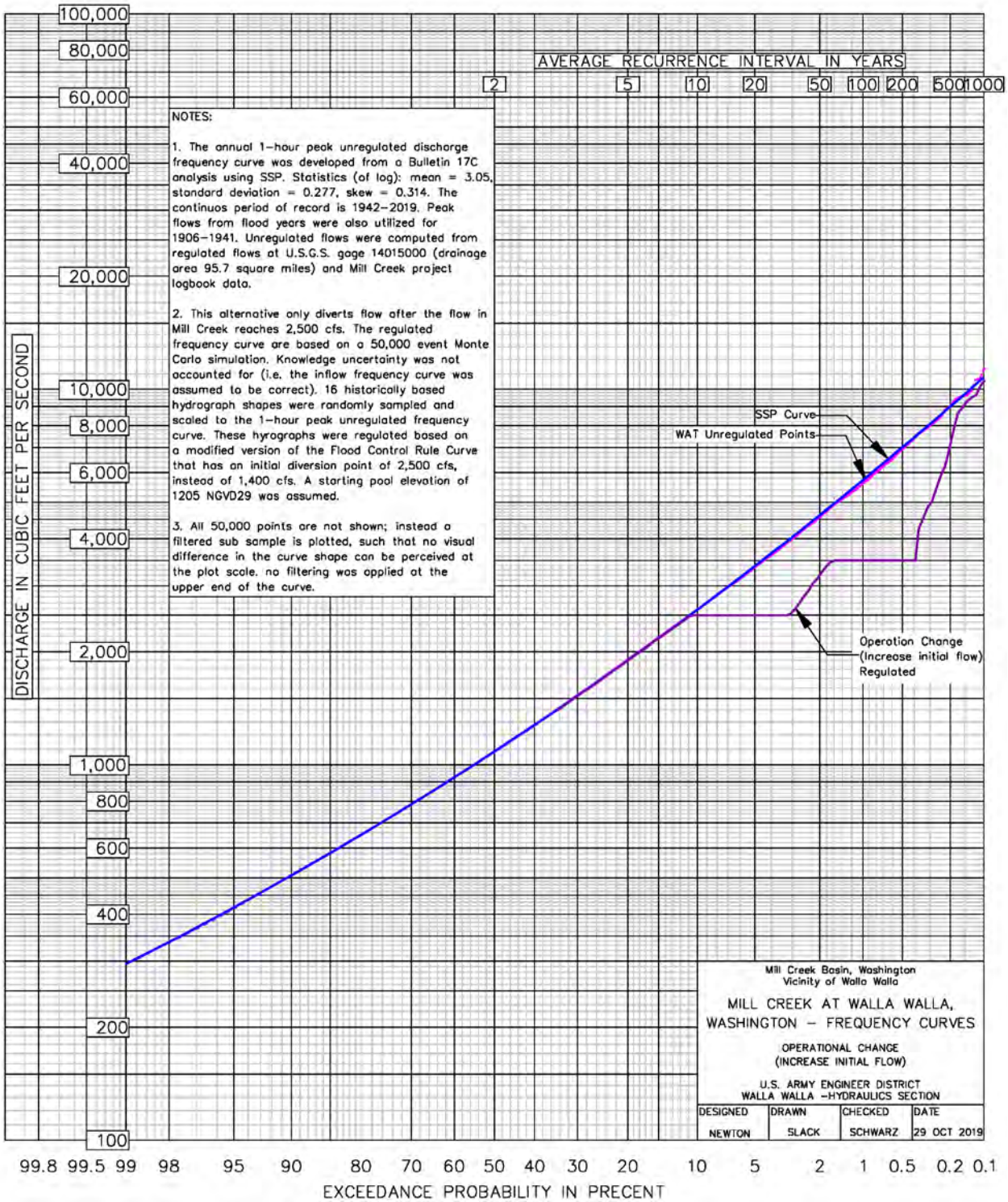


Figure 3-24. Operational Change Regulated Flow Frequency Curve

This operation maintains flows to the channel flow target of 3,500 cfs up to the 0.39-percent chance flood (255 Year).

The regulated frequency curve was entered into HEC-FDA as a graphical frequency curve. The following data points were input into the HEC-FDA model with a period of record of 78 years.

Table 3-10. Operations Change Regulated Frequency Curve FDA Input

Annual Exceedance Probability	Discharge (cfs)
0.9952	261
0.9828	327
0.9536	406
0.9	510
0.6	927
0.5	1081
0.4104	1249
0.3471	1399
0.102138	2500
0.032399	2501
0.02	3200
0.01	3500
0.00392	3500
0.0039	3850
0.00222	6197
0.0017	8680
0.001	10604
0.00018	16960
0.0001	19670

From these points, HEC-FDA generated the following frequency curve data and plot.

DRAFT

Function: operation change

Use An Existing Function

Save

Cancel

Description:

Type

Analytical...

Graphical...

Function Statistics...

Plot...

Exceedance Probability	Discharge (cfs)	Confidence Limit Curves Discharge (cfs) -2 SD	-1 SD	+1 SD	+2 SD
0.99900	259.55	172.31	211.48	318.55	390.96
0.99000	295.81	196.39	241.03	363.05	422.43
0.95000	414.42	335.93	373.12	460.30	511.26
0.90000	510.00	413.49	459.22	566.40	629.04
0.80000	658.57	539.58	596.11	727.58	803.81
0.70000	791.90	663.18	724.68	865.34	945.60
0.50000	1,081.00	907.06	990.22	1,180.11	1,288.30
0.30000	1,526.08	1,251.96	1,382.24	1,684.89	1,860.22
0.20000	1,882.98	1,518.64	1,691.03	2,096.73	2,334.74
0.10000	2,500.02	2,499.28	2,499.65	2,500.39	2,500.76
0.04000	2,500.83	2,499.95	2,500.39	2,501.27	2,501.72
0.02000	3,200.00	2,500.30	2,523.42	3,761.08	4,244.81
0.01000	3,500.00	2,885.61	3,177.99	3,854.64	4,245.20
0.00500	3,500.75	2,886.23	3,178.67	3,855.46	4,246.11
0.00400	3,500.98	2,886.42	3,178.88	3,855.71	4,246.39
0.00200	7,076.75	5,834.50	6,425.67	7,793.79	8,583.49
0.00100	10,604.00	8,742.58	9,628.41	11,678.44	12,861.75
0.00010	19,669.99	16,217.12	17,860.32	21,663.03	23,858.03

Figure 3-25. Operations Change Regulated Frequency Curve FDA Extrapolation

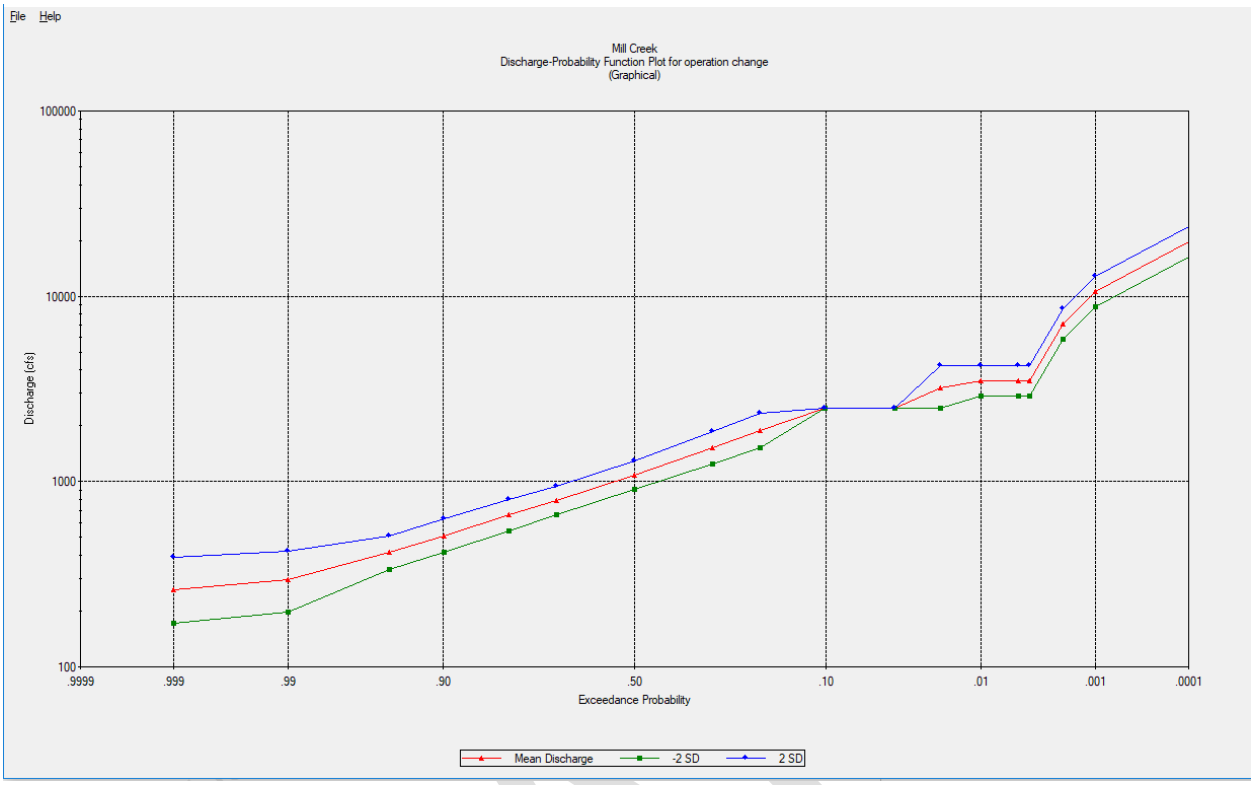


Figure 3-26. Operations Change Regulated Frequency Curve in FDA

Note that HEC-FDA extracts values from the entered data for predetermined selected frequencies. This hard fix in the model software diminishes the resolution and reduces the preciseness of the results.

HEC-FDA does not provide the expected uncertainty breadth in the frequency curve when there are areas of small increases in flows with diminishing probabilities, when using the graphical frequency curve approach. We do not see the expected spread in uncertainty as you move from the 1% flood to the 0.1% flood. These are limitations in HEC-FDA software that need to be understood in the context of this study. HEC was consulted on these issues and an alternative method will be used for the final model runs that inputs the unregulated frequency curve and then uses a transform function to regulate. This should better define the confidence limits.

<u>Regulated Frequency Curve Comparisons</u>

The sampled unregulated frequency curve along with the resulting regulated frequency curve for each alternative are shown in Figure 3-27.

The operational change alternative only was carried through the regulated frequency curve development.

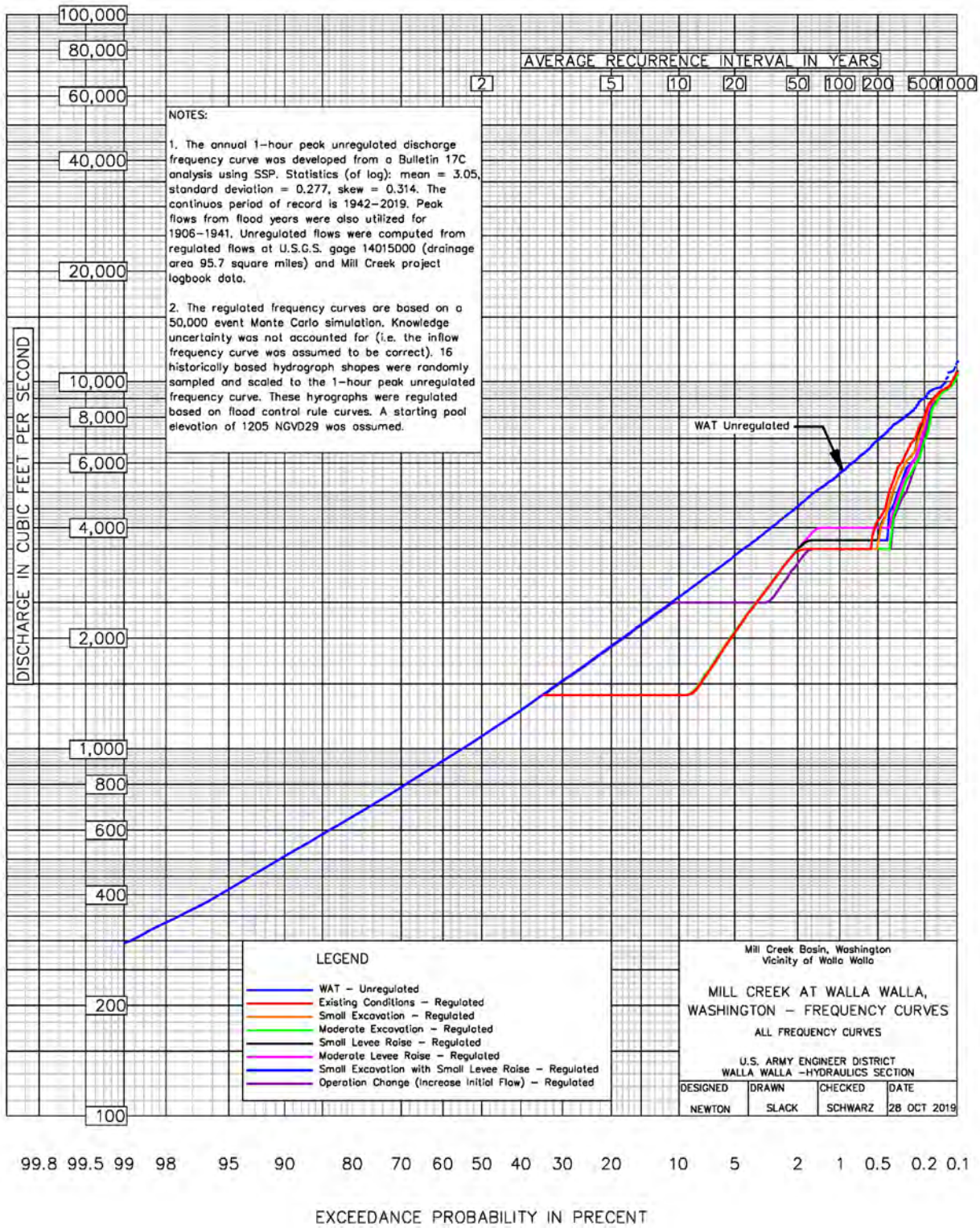


Figure 3-27. Frequency Curve Alternative Comparison

3.3 INUNDATION MODELING

3.3.1 HEC-RAS model Development

Terrain Data

The HEC-RAS model used digital orthophoto and LiDAR data provided by Rogers Surveying Inc., P.S. in February 2010. This work was completed under contract with the U.S. Army Corps of Engineers, Walla Walla District, Contract No. W912EF-08-D-0011, Task Order No. 18. The data was provided in horizontal datum US State Plane, Washington South Zone, NAD'83; vertical datum is NAVD'88, unit of measure is US survey foot. The orthophoto accuracy requirement was: horizontal = Root Mean Square Error (RMSE) 5 feet. The LiDAR accuracy requirement was: horizontal = RMSE 0.6 feet, and vertical = RMSE 0.33 feet.

The LiDAR data showed very good correlation to quality control points. A total of 634 quality control points were analyzed, the mean difference between them and the LiDAR is 0.28 feet, and the RMSE is 0.36 feet. The best correlations were areas with bare earth or concrete, differences here were typically 0.1 foot. Areas with grass and other vegetation cover had similar differences to the mean, and up to 1.5 feet with extremely heavy vegetation. Overall, the quality control verification provides strong confidence in the LiDAR data.

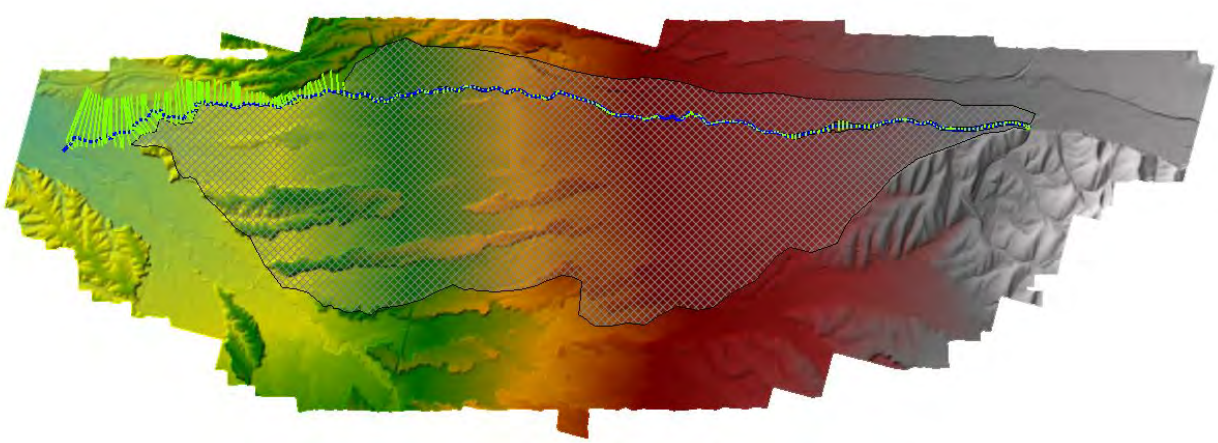


Figure 3-28. LiDAR Data

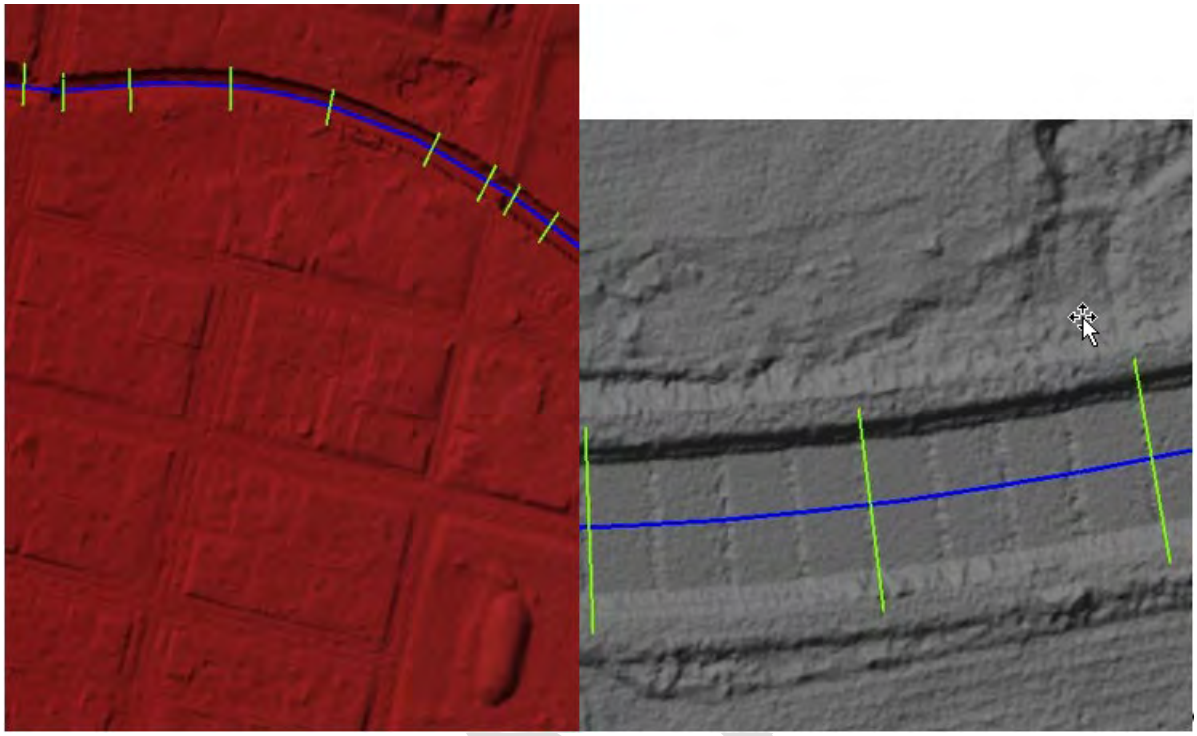


Figure 3-29. LiDAR Data Close-Ups
(Left Image) City Blocks with streets visible and buildings/vegetation removed
(Right Image) Levee section with channel stabilizers visible

Model Set Up

HEC-RAS version 5.0.6 (November 2018) was used. The horizontal datum was "USA_Contiguous_Albers_Equal_Area_Conic_USGS_version, Foot_US". The vertical datum was "NAVD88" in units of feet.

Mill Creek has no inflowing creeks within the model extents. Russel Creek and Yellowhawk Creek flow from Mill Creek but were not modeled because their flows are 30 cfs or less and do not take a significant amount of flow from Mill Creek. The HEC-RAS model was created in 1D for the channel and 2D for the overbanks. To capture flow spreading out into the basin the 2D grid had to be cover a large area. The 2D gridded area was arranged in 50 x 50 ft. cells. The cell size had to be small enough for water to be seen flowing along streets or inundating the buildings above the streets. This 50 ft. cell size allowed enough precision for calculating flow velocity and direction while still keeping computer run times reasonable.

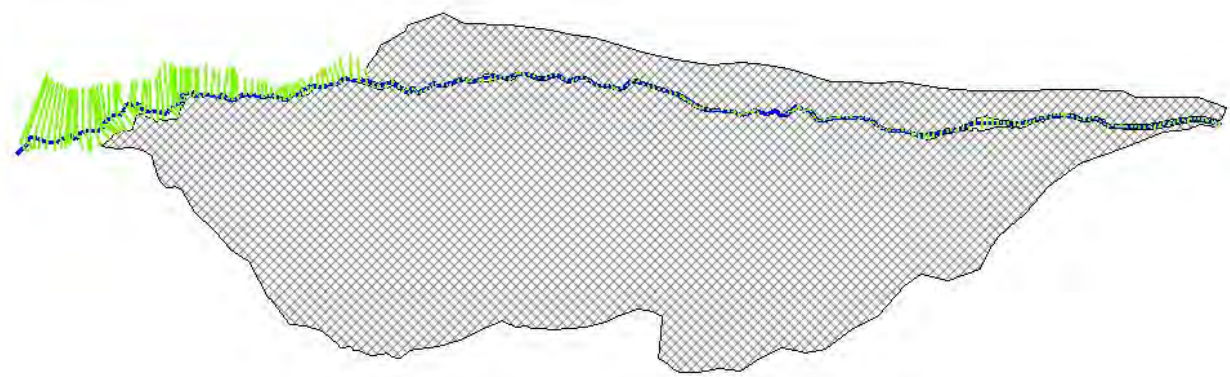


Figure 3-30. 1D River Channel with Cross Sections, 2D Grid in Overbanks

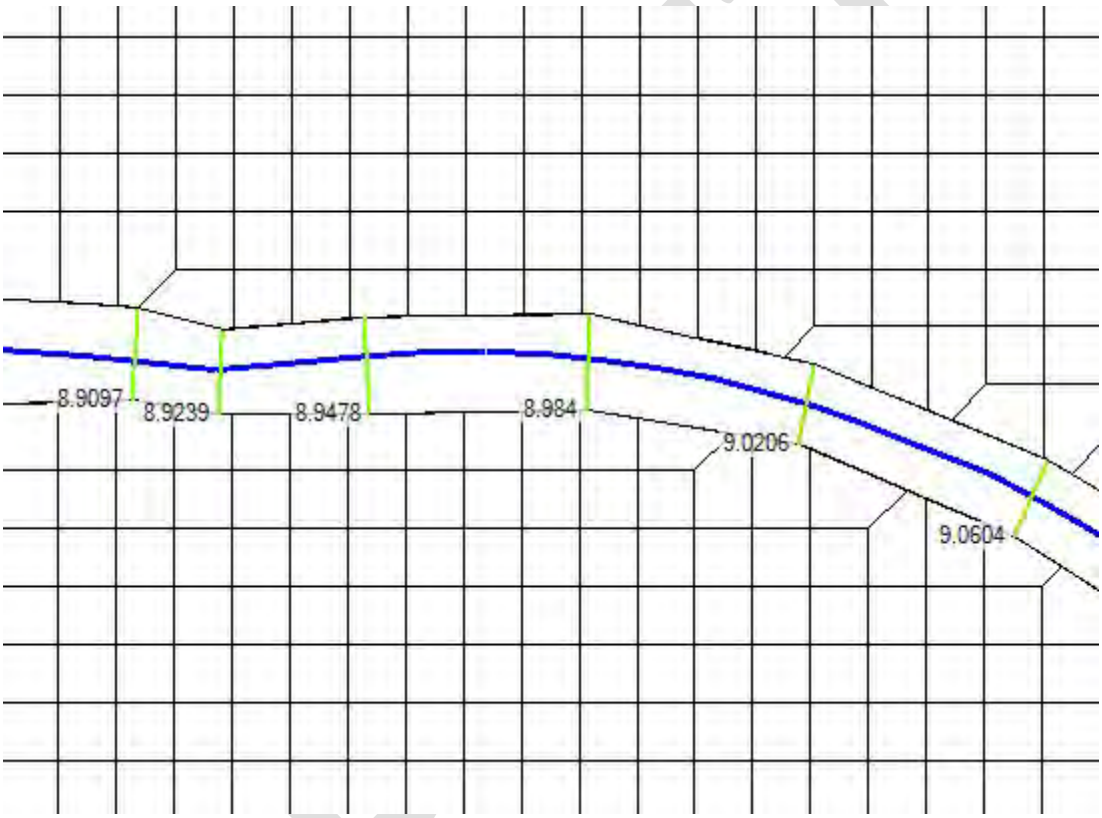


Figure 3-31. Close Up of 1D River Channel with Cross Sections, 2D Grid in Overbanks

There have not been any surveys for Mill Creek or its bridges, only design plans. The channel and bridge geometry was taken from design plans furnished by the Army Corps of Engineers and the City of Walla Walla. Cross sections were placed directly over the stabilizers in the stabilizer/levee sections. The channel bottom elevations were taken from the tops of the stabilizers which were in the design plans.

Levees were modeled as lateral structures. Lateral structures where the dividing line between the 1D and 2D areas of the model. The National Levee Database (NLD) had levee centerline and

elevation data. It was unknown if the NLD elevations were updated after levee maintenance projects which affected the levee heights. LiDAR was used instead of the NLD for the levee elevations. The NLD centerline alignment was used so that the NLD data could be updated in the future.

The concrete channel geometry through the city of Walla Walla was taken from design plans. There have been some modifications to the original concrete for fish passage projects which were incorporated in the channel geometry. In a few places such as bridge designs the channel width was identified, but the channel width was largely measured from aerial imagery. Channel wall heights were scaled off design plans. Little was known about the tunnel going underneath downtown Walla Walla. The original plans were lacking in detail. The tunnel widths, heights, construction materials, and flow capacity were largely unknown. The tunnel was modeled as a non-pressurized culvert that did not limit the overall capacity.

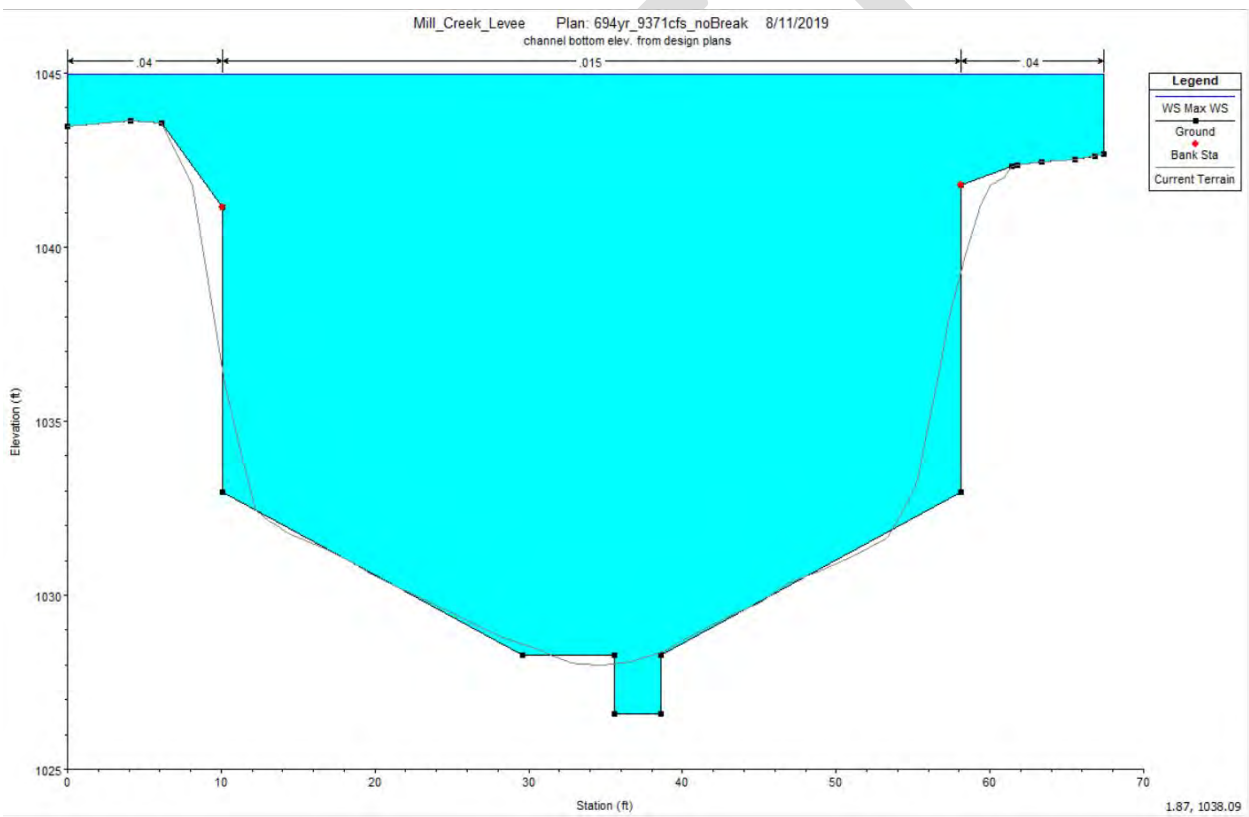


Figure 3-32. Typical Cross Section in Concrete Channel Section

The Mill Creek 1D sections with stabilizers/levees had a Manning's n value of 0.04 in the channel and 0.04 in the levees. The concrete sections in town had a Manning's n value of 0.015 in the channel and walls and 0.04 in the overbanks. Downstream of the city Mill Creek goes back to a natural channel with a Manning's n value of 0.035 in the channel and 0.04 to 0.055 in the overbanks. The Mill Creek 2D grid also received Manning's n values. Shapefiles of the roads, buildings, sidewalks, forest, crops etc. were laid over the 2D area. These shapefiles were obtained from national databases, the City of Walla Walla, the City of College Place, and Walla Walla County. Manning's n values were assigned to the different shapefiles.

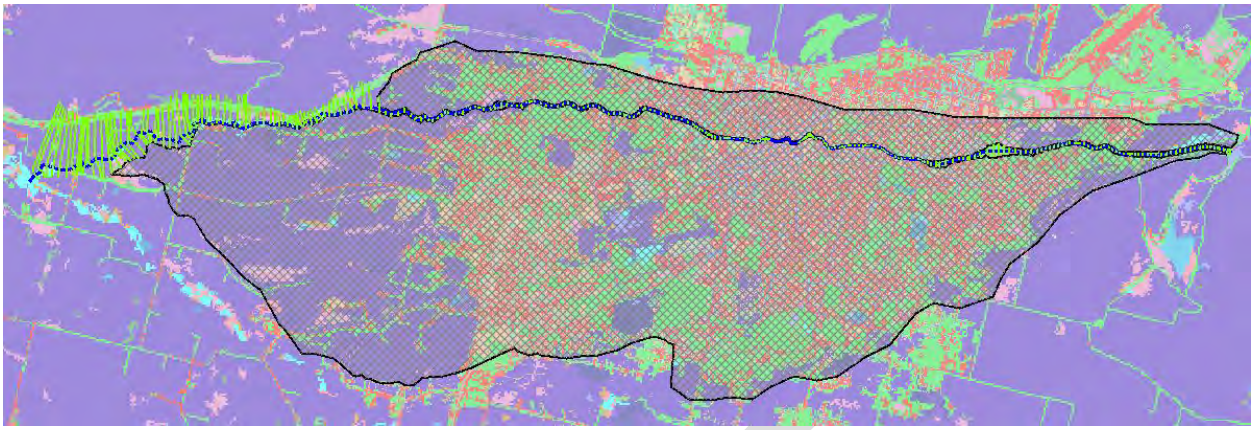


Figure 3-33. Manning's n Values

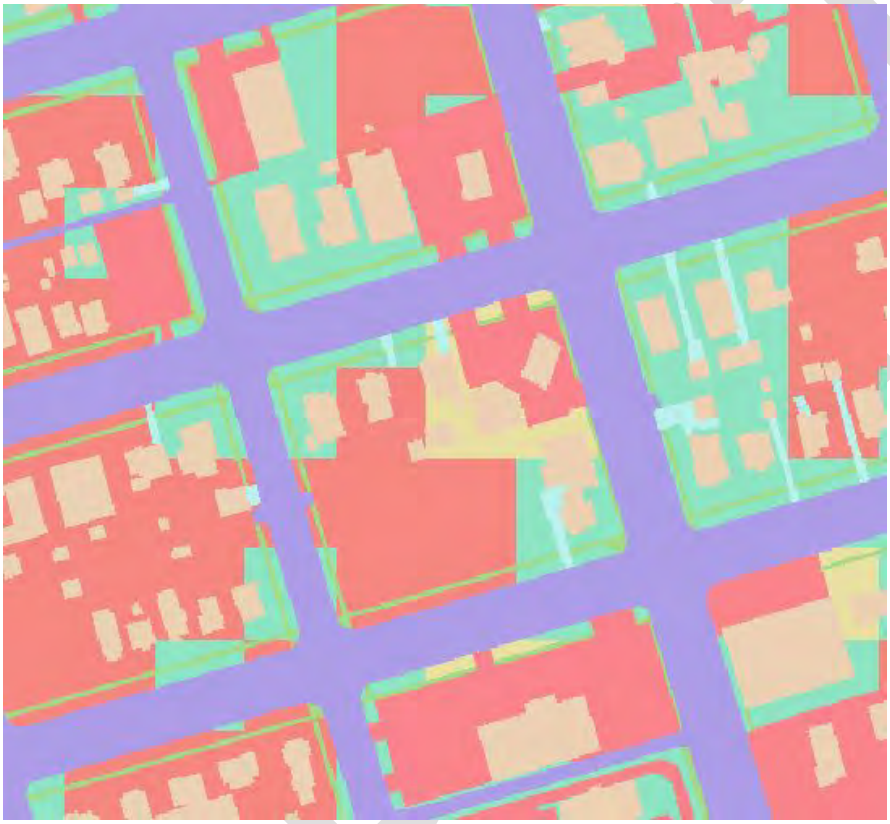


Figure 3-34. Manning's n Values Close-Up

Table 3-11. Manning's n Values Used in 2D Areas

		Manning's n
	Building	99
	Road	0.016
	Road Shoulder	0.016
	Sidewalk	0.015
	Parking Lot	0.015
	Driveway	0.016
National Land Cover Database (NLCD)		**Manning's n**
11	Open Water	0.038
21	Developed, Open Space	0.040
22	Developed, Low Intensity	0.100
23	Developed, Medium Intensity	0.110
24	Developed, High Intensity	0.160
31	Barren Land (Rock, Sand, Clay)	0.027
41	Deciduous Forest	0.130
42	Evergreen Forest	0.130
43	Mixed Forest	0.130
52	Shrub/Schub	0.110
71	Grassland/Herbaceous	0.110
81	Pasture/Hay	0.037
82	Cultivated Crops	0.037
90	Woody Wetlands	0.090
95	Emergant Herbaceous Wetlands	0.067

Note: If occupying the same space, objects higher on the list override objects lower on the list

As water flowed overland and encountered the different n values assigned to the various shapefiles the water would flow faster or slower. Faster water has less depth than slower water in the same area. As expected, the model showed water flowing fast down roads and much slower when it encountered buildings or other obstructions.

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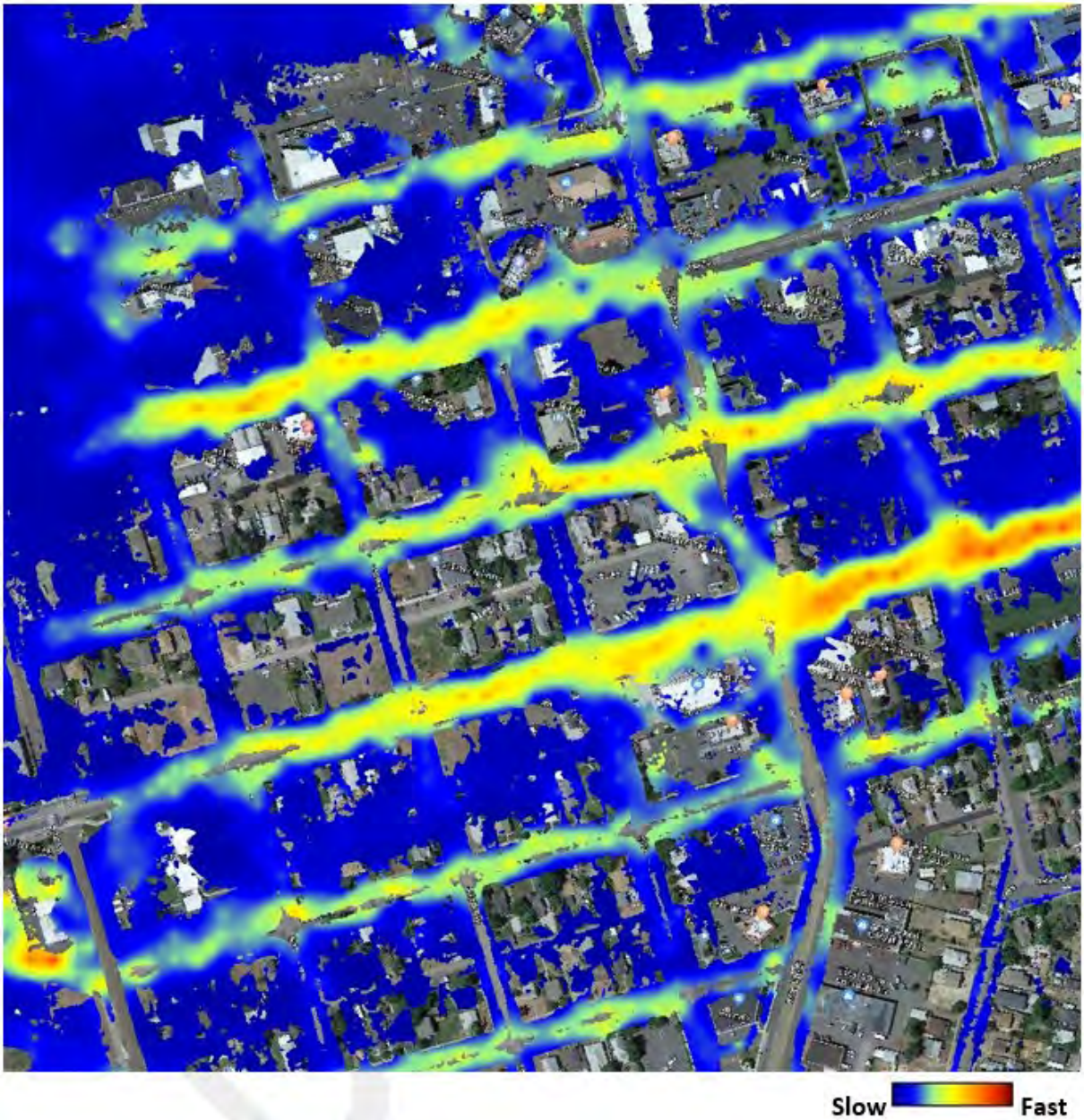

Figure 3-35. Velocity Map

<u>Model Runs</u>

The HEC-RAS model was run with 6 different scenarios:

- Existing Conditions
- Small Excavation
- Moderate Excavation
- Small Levee Raise

- Moderate Levee Raise
- Small Excavation plus Small Levee Raise

Each of the 6 scenarios had 8 to 9 hydrograph flows with varying probabilities. See table below.

Table 3-12. Flows for 6 Scenarios

Existing Conditions									
Return Period (yr):	1000	694	543	431	248	56 to 176	46	20	11
Probability:	0.001	0.00144	0.00184	0.00232	0.00404	0.01758 to 0.00568	0.02194	0.05	0.0913
Peak Flow (cfs):	10604	9371	8640	7171	5005	3500	3337	2069	1400
Small Excavation									
Return Period (yr):	1000	694	538	442		57 to 176	46	20	11
Probability:	0.001	0.00144	0.00186	0.00226		0.01758 to 0.00506	0.02194	0.05	0.0913
Peak Flow (cfs):	10544	9371	8476	6803		3500	3337	2074	1400
Moderate Excavation									
Return Period (yr):	1000	704		450	262	57 to 251	46	20	11
Probability:	0.001	0.00142		0.00222	0.00382	0.01758 to 0.00398	0.02194	0.05	0.0913
Peak Flow (cfs):	10311	9265		6162	4165	3500	3340	2088	1400
Small Levee Raise									
Return Period (yr):	1000	694	588	442	424	65 to 200	46	20	11
Probability:	0.001	0.00144	0.0017	0.00226	0.00236	0.0153 to 0.005	0.02194	0.05	0.0913
Peak Flow (cfs):	10604	9371	8875	7273	6494	3700	3337	2069	1400
Moderate Levee Raise									
Return Period (yr):	1000	694	526	455		76 to 250	46	20	11
Probability:	0.001	0.00144	0.0019	0.0022		0.0132 to 0.004	0.02194	0.05	0.0913
Peak Flow (cfs):	10604	9371	8311	6517		4000	3337	2069	1400

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Small Excav. + Small Levee Raise									
Return Period (yr):	1000	694	581	439	244	65 to 239	46	20	11
Probability:	0.001	0.00144	0.00172	0.00228	0.0041	0.0153 to 0.00418	0.02194	0.05	0.0913
Peak Flow (cfs):	10544	9342	8778	6432	4225	3700	3337	2074	1400

For each of the 6 scenarios and each of the 8-9 hydrographs HEC-RAS had the following 5 Model Runs:

- No Break – No levee breaks or channel blockages, just overtopping
- Yellowhawk Levee Break – Left side levee failure if overtopped, near Yellowhawk Creek (break centered at Mill Creek cross section 11.5168)
- Tausick Way Levee Break – Left side levee failure if overtopped, near Tausick Way (road) (break centered at Mill Creek cross section 10.9688)
- Otis Street Blockage – 50% blockage of Mill Creek concrete channel near Otis Street (Mill Creek station 8.68)
- Tunnel Blockage – 90% blockage of the tunnel (Mill Creek station 8.183)

The total possible HEC-RAS runs was (6 scenarios) X (9 hydrographs) X (5 Model Runs) = 270 runs. Not all the runs were necessary. Runs with a peak flow less than 4500 cfs would not overtop and therefore reduced the number of runs. Some runs had the same geometry files with similar hydrographs and wouldn't have to be modeled. The total number of runs necessary was 162. It took about 800 hours for HEC-RAS to complete all the runs. Multiple computers running HEC-RAS were used at the same time.

The model 1D channel was processed in 30 second time steps and the 2D grid was processed in 4 second time steps. It took about 16 hours for water to spread out fully and reach the bottom of the 2D area. In a 2D area setting a Courant number equal to 1 is a way to estimate what the time step should be with a given grid size and flow velocity. The 4 second time step was calculated from using the Courant number equal to 1. The 4 second time step was further reduced if the model failed to run or became unstable.

The Mill Creek floodplain is an alluvial fan. Once water escapes its banks it spreads out quickly. Flow escaping from the left bank (looking downstream) either went south-west toward the Walla Walla River or went west and rejoined Mill Creek further downstream. Flow escaping from the right bank rejoined Mill Creek further downstream.

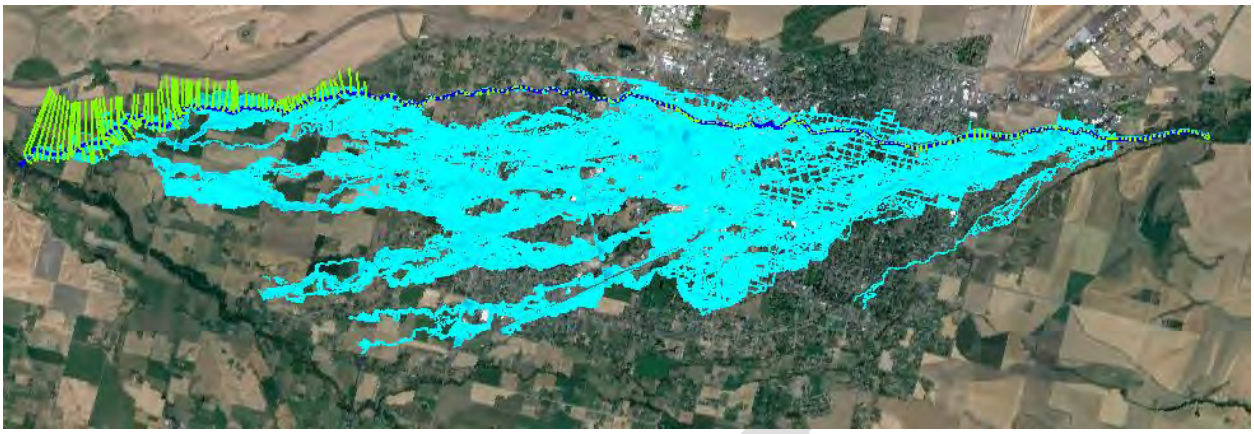


Figure 3-36. Mill Creek Flood Flows – Alluvial Fan

Depth grids were made from the maximum depth of each cell. These depth grids were used in the HEC-FDA program as part of the economic analysis.

3.3.2 Levee Sizing for Levee Raise and Certification

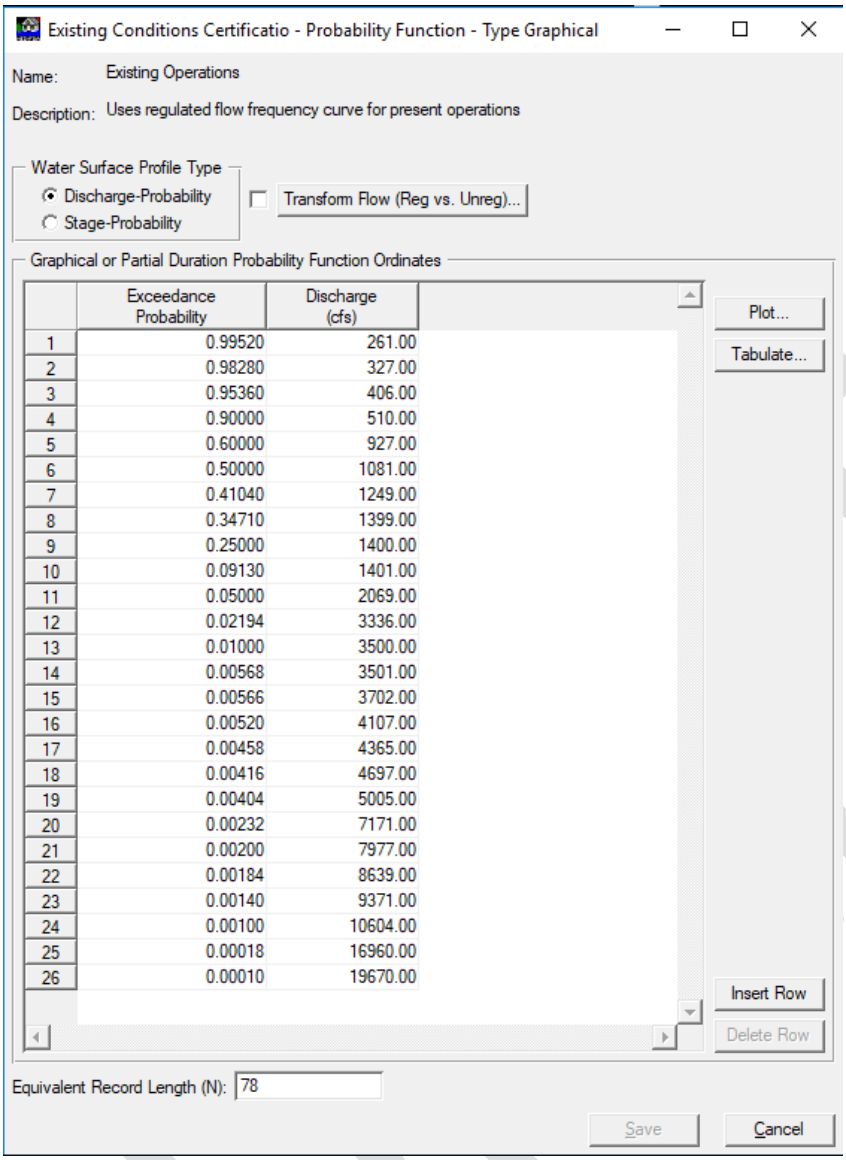
It was determined in section 2.2.3 that the stabilizers would be able to accommodate higher flows than 3,500 cfs without expected failure. Based on that levee raises were considered as an alternative for this study. This section discusses the approach, analysis, and assumptions that went into evaluating existing levees for levee certification as well as sizing levees for levee raise alternatives that meet levee certification standards. This analysis follows hydrology and hydraulic requirements outlined in ECB 2019-11 (EC 1165-2-218 Draft), three options were considered: the present 3,500 cfs 1-percent exceedance flood, a future levee raise targeting a 3,700 cfs 1-percent exceedance flood, and a future levee raise targeting a 4,000 cfs 1-percent exceedance flood. The primary tool used for this analysis was HEC-FDA.

3.3.2.1 Existing Conditions – Present Operations

To evaluate the existing conditions to meet hydrology and hydraulic characteristics required for levee certification by ECB 2019-11 (EC 1165-2-218 Draft) the following analysis and assumptions were made.

<u>Flow Frequencies</u>

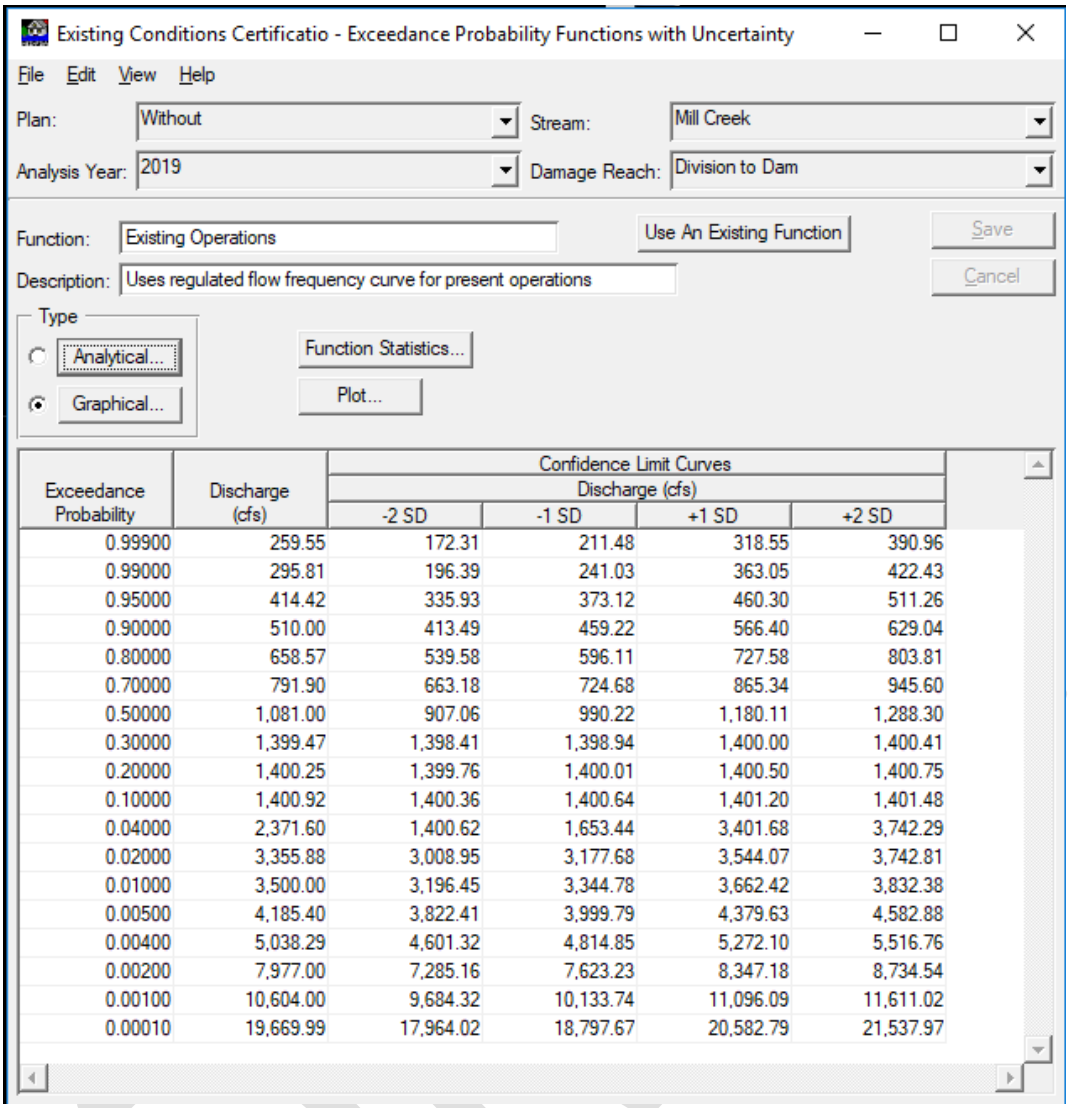
Existing conditions regulated frequency curve was placed into HEC-FDA as a graphical frequency curve. The regulated frequency curve was developed as documented in " Regulated Flow Frequency Curve Development" The following data points were input into the model with a period of record of 78 years.



	Exceedance Probability	Discharge (cfs)
1	0.99520	261.00
2	0.98280	327.00
3	0.95360	406.00
4	0.90000	510.00
5	0.60000	927.00
6	0.50000	1081.00
7	0.41040	1249.00
8	0.34710	1399.00
9	0.25000	1400.00
10	0.09130	1401.00
11	0.05000	2069.00
12	0.02194	3336.00
13	0.01000	3500.00
14	0.00568	3501.00
15	0.00566	3702.00
16	0.00520	4107.00
17	0.00458	4365.00
18	0.00416	4697.00
19	0.00404	5005.00
20	0.00232	7171.00
21	0.00200	7977.00
22	0.00184	8639.00
23	0.00140	9371.00
24	0.00100	10604.00
25	0.00018	16960.00
26	0.00010	19670.00

Figure 3-37. Existing Conditions Regulated Frequency Curve FDA Input

From these points, HEC-FDA generated the following frequency curve table and plot.



Exceedance Probability	Discharge (cfs)	Confidence Limit Curves Discharge (cfs) -2 SD	-1 SD	+1 SD	+2 SD
0.99900	259.55	172.31	211.48	318.55	390.96
0.99000	295.81	196.39	241.03	363.05	422.43
0.95000	414.42	335.93	373.12	460.30	511.26
0.90000	510.00	413.49	459.22	566.40	629.04
0.80000	658.57	539.58	596.11	727.58	803.81
0.70000	791.90	663.18	724.68	865.34	945.60
0.50000	1,081.00	907.06	990.22	1,180.11	1,288.30
0.30000	1,399.47	1,398.41	1,398.94	1,400.00	1,400.41
0.20000	1,400.25	1,399.76	1,400.01	1,400.50	1,400.75
0.10000	1,400.92	1,400.36	1,400.64	1,401.20	1,401.48
0.04000	2,371.60	1,400.62	1,653.44	3,401.68	3,742.29
0.02000	3,355.88	3,008.95	3,177.68	3,544.07	3,742.81
0.01000	3,500.00	3,196.45	3,344.78	3,662.42	3,832.38
0.00500	4,185.40	3,822.41	3,999.79	4,379.63	4,582.88
0.00400	5,038.29	4,601.32	4,814.85	5,272.10	5,516.76
0.00200	7,977.00	7,285.16	7,623.23	8,347.18	8,734.54
0.00100	10,604.00	9,684.32	10,133.74	11,096.09	11,611.02
0.00010	19,669.99	17,964.02	18,797.67	20,582.79	21,537.97

Figure 3-38. Existing Conditions Regulated Frequency Curve FDA Extrapolation

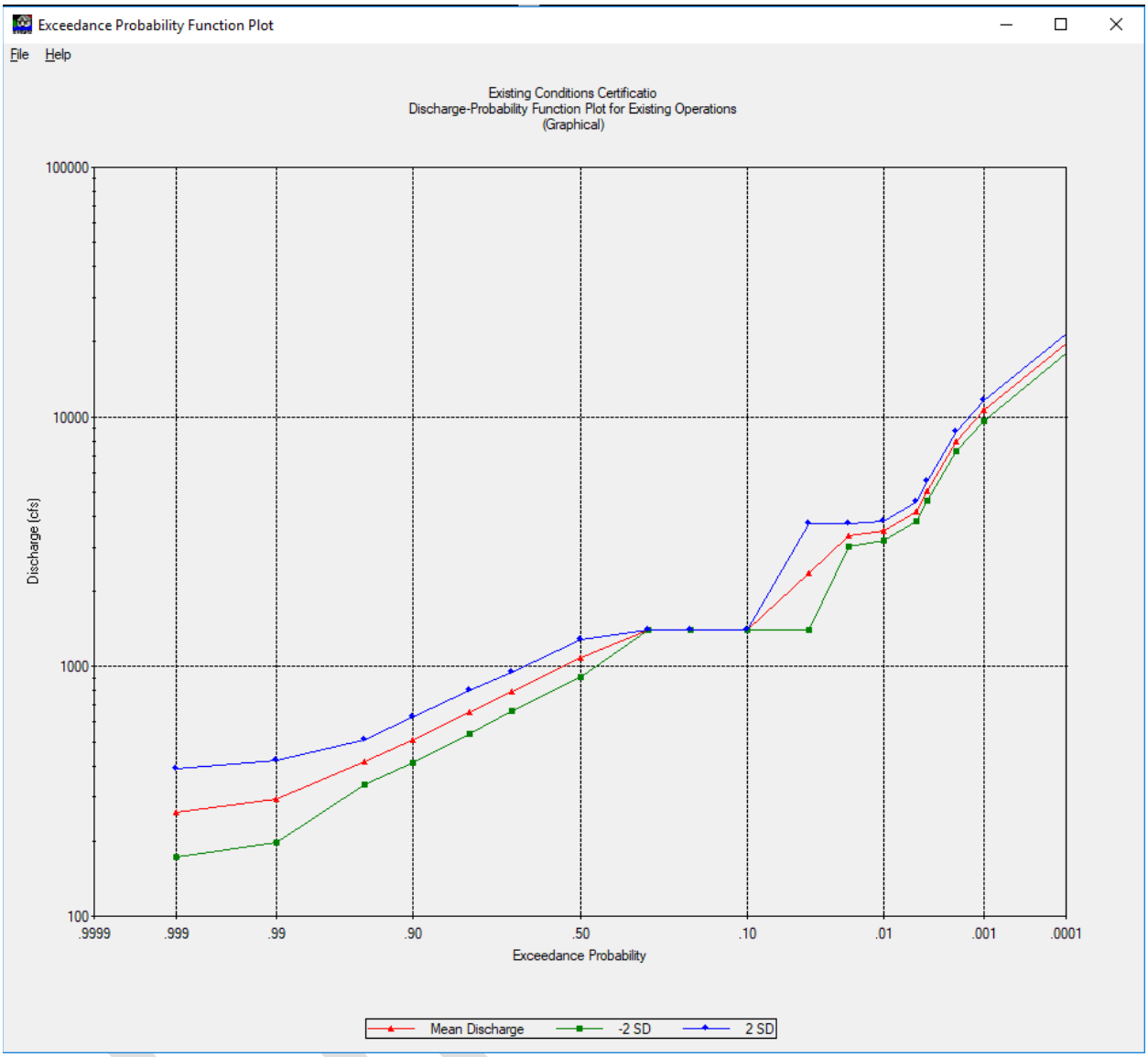


Figure 3-39. Existing Conditions Regulated Frequency Curve in FDA

Note that HEC-FDA extracts values from the entered data for predetermined selected frequencies. This hard fix in the model software diminishes the resolution and reduces the preciseness of the results.

HEC-FDA does not provide expected uncertainty breadth in the frequency curve when there are areas of small increases in flows with diminishing probabilities, when using the graphical frequency curve approach. This is the reason that the frequency curve confidence bounds narrow particularly for the 1,400 cfs regulation target and to a lesser degree for the 3,500 cfs regulation target. We also do not see the expected spread in uncertainty as you move to the 0.1% flood. These are limitations in HEC-FDA software that need to be understood in the context of this study. HEC was consulted on these issues and an alternative method will be used for the final model runs that inputs the unregulated frequency curve and then uses a transform function to regulate. This should better define the confidence limits.

Hydraulic Model

For evaluation for levee certification the unsteady hydraulic model developed for inundation mapping was adapted to be a steady flow model. The 2D overbank areas were removed and the model was utilized as a steady flow 1D channel only model for levee sizing and certification. Eight steady flow profiles were developed for the for specific percent exceedance flows as shown the following table. These were developed at key points in the regulated frequency curve.

Table 3-13. Existing Conditions Frequency Flows, Levee Sizing

Probability	Flow (cfs)
0.25	1,400
0.05	2,069
0.02194	3,336
0.01	3,500
0.00404	5,005
0.00232	7,171
0.00184	8,639
0.001	10,604

Channel n values were set to 0.04 for the levee reach.

Evaluation Reaches

In the leveed areas, evaluation reaches were identified between selected bridges. Within the reaches index locations were selected based on the least available freeboard between the 1 percent chance flood profile (3,500 cfs) and the levee top. Depending on available freeboard anywhere from 1 to 3 index locations were selected for each reach. The following table indicates the reaches, model reach names in HEC-FDA and index cross sections from the RAS model.

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Table 3-14. Evaluation Reaches for Levee Sizing

Reach	Approximate Reach Length (feet)	Reach Name in FDA	Index Cross Section in RAS and FDA
Division Works to Dam	4,880	Division to Dam	12.2216
		Division to Dam -1	11.5168
Tausick to Division Works	2,890	Tausick to Division Works2	11.0901
		Tausick to Division Works	10.9688
Wilber to Tausick	5,600	Roosevelt to Tausick2	10.4022
		Roosevelt to Tausick	10.3597
Roosevelt to Wilber	2,200	Roosevelt to Tausick -1	9.8223
Myra to 9th	5,460	Myra to 9th 3	7.3084
		Myra to 9th 2	7.1731
		Myra to 9th	6.4828
Gose to Myra	4,870	Gose to Myra 3	6.3902
		Gose to Myra 2	6.2838
		Gose to Myra	5.607

<u>Stage Uncertainty</u>

In the HEC-FDA model, the Standard Deviation of the Error was determined by computation of uncertainty in friction and wave height. The manning's n values were varied in the HEC-RAS model to determine the standard deviation for the index locations for each flow frequency. The low channel manning's n was set to 0.03 and the high manning's n was set to 0.05, with the best estimate being 0.04. The high stage was further adjusted by 1.5 foot for wave action as observed at stabilizers near the levee during the 1996 flood. The following equation was used to determine the Standard deviation of error for each flows surrounding the design target at index cross sections.

Standard Deviation of the Error = [(high stage – expected stage)^2 + (low stage – expected stage)^2 /2]$^{1/2}$

Next, the equation results were compared to Table 5-2 in EM 1110-2-1619 to assure that these values met minimum standard deviation of error for stage as recommended in the EM. For these reaches the table indicated something between 0.7 and 1.3 would be a minimum value. Finally, to get smoothed results a single value was used for all flood frequencies at each index location based on evaluation of the calculated Standard Deviation of the Error and the EM guidance. This value was selected from flows between 3,500 cfs and 5,000 cfs as this is the critical flow range for levee sizing and certification. The reason for selecting a single value was based on lower confidence in the values obtained from the rough hydraulic model where instabilities have not all been worked out, and meeting the minimum range given in the EM. The following table includes the calculated values from the Standard Deviation of the Error

Equation for the critical flow range used for levee sizing and levee certification calculations and the selected value.

Table 3-15. Standard Deviation of the Error for Index Cross Sections

Index Cross Section	Flow (cfs)	Calculated Standard Deviation of the Error	Selected Standard Deviation of the Error for Index Cross Section
12.2216	3,500	1.418678963	1.5
	5,005	1.523039724	
11.5168	3,500	1.060683742	1.1
	5,005	1.075174404	
11.0901	3,500	1.330883165	1.3
	5,005	1.324178991	
10.9688	3,500	1.273184983	1.3
	5,005	1.267142455	
10.4022	3,500	1.23004065	1.3
	5,005	1.315028517	
10.3597	3,500	1.364734406	1.4
	5,005	1.329529992	
9.8223	3,500	1.370638537	1.6
	5,005	1.563105883	
7.3084	3,500	1.060660172	1.1
	5,005	1.075011628	
7.1731	3,500	1.067754653	1.1
	5,005	1.089311709	
6.4828	3,500	1.431083506	1.5
	5,005	1.512696268	
6.3902	3,500	1.396513516	1.4
	5,005	1.439183796	
6.2838	3,500	1.380036231	1.4
	5,005	1.429772709	
5.607	3,500	1.60338704	1.7
	5,005	1.680371983	

<u>Results</u>

To meet levee certification requirements for hydrology and hydraulics outlined in ECB 2019-11 (EC 1165-2-218 Draft), a levee needs a Conditional Non-Exceedance Probability of greater than 0.85 for the 1% event at all index locations.

Analysis indicates that these levees in their present condition with present reservoir operations fall marginally below requirements for levee certification for the existing conditions 1 percent chance flood flow of 3,500 cfs in one reach. The following table summarizes the results

Table 3-16. Existing Conditions Required Levee Raise

Critical Index Cross Section	Damage Reach	Conditional Non-Exceedance Probability 1% Exceedance Flood (3,500 cfs)	Levee Raise Required to meet 0.85 Probability for Levee Certification (feet)
12.2216	Division Works to Dam	0.982	0.0
10.9688	Tausick to Division Works	0.918	0.0
10.3597	Wilber to Tausick	0.873	0.0
9.8223	Roosevelt to Wilber	0.979	0.0
7.3084	Myra to 9th	0.842	0.2
5.607	Gose to Myra	0.891	0.0

To meet certification requirements the levee from Myra to 9th Street Bridges will need to be raised 0.2 feet.

As the hydraulic model and hydrology are refined a more definitive determination can be made. However, these results do indicate that there is a good chance the levees will not meet hydraulic requirements for levee certification and a minimal raise will be required.

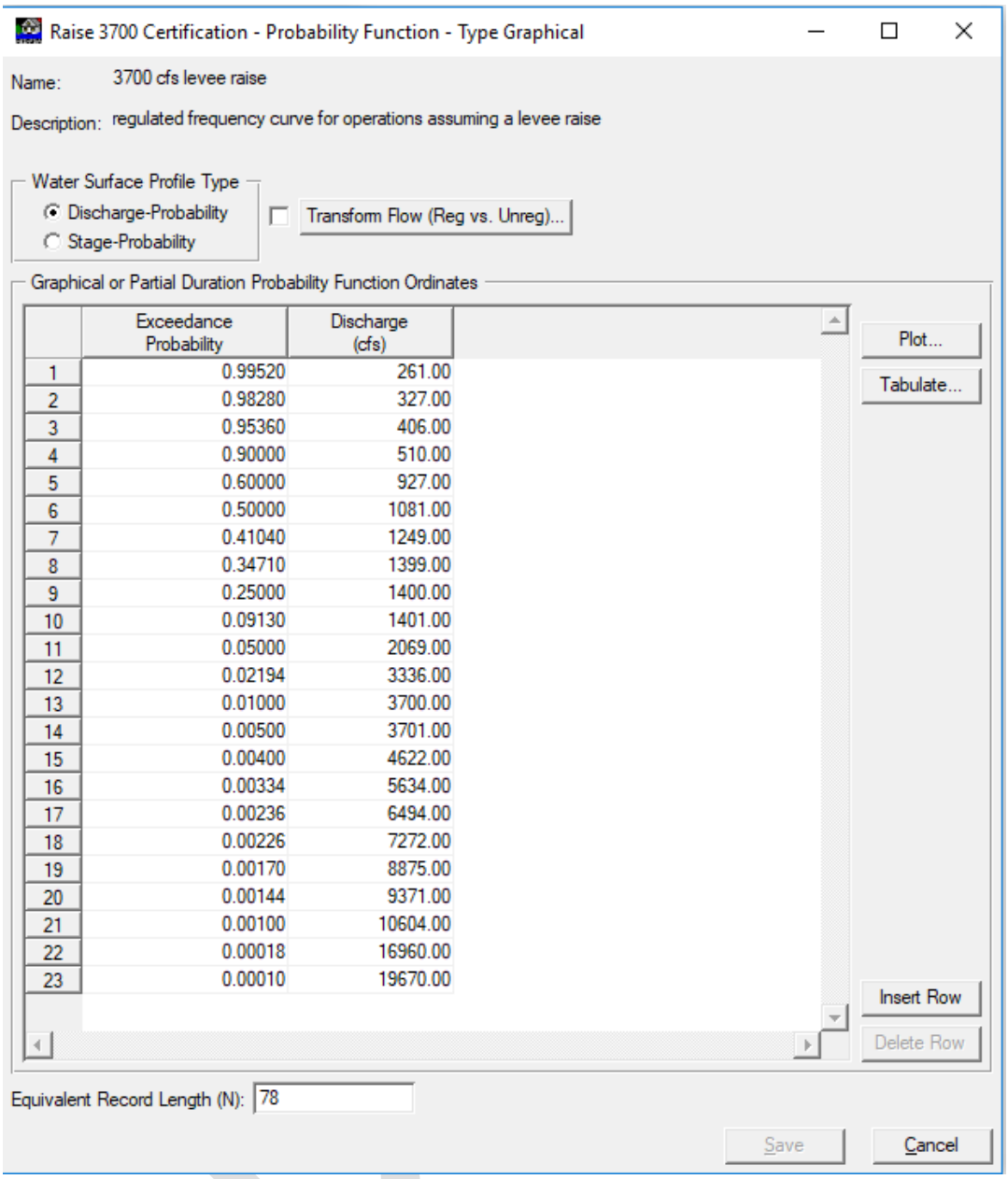
3.3.2.2 Small Levee Raise Alternative

For this alternative the regulated flow frequency curve was revised to change the regulation target from 3,500 cfs to 3,700 cfs. The regulated frequency curve under this alternative shows the 1-percent chance flood as 3,700 cfs. The rest of this section outline levee sizing to meet levee certification requirements for a 3,700 cfs levee.

<u>Flow Frequencies</u>

The 3,700 cfs Levee Raise Alternative regulated frequency curve was placed into HEC-FDA as a graphical frequency curve. The regulated frequency curve was developed as documented in " Regulated Flow Frequency Curve Development" This frequency curve is unchanged from the existing conditions frequency curve below 3,400 cfs (~0.02 percent flood) and above 10,000 cfs (~ 0.001 percent flood).

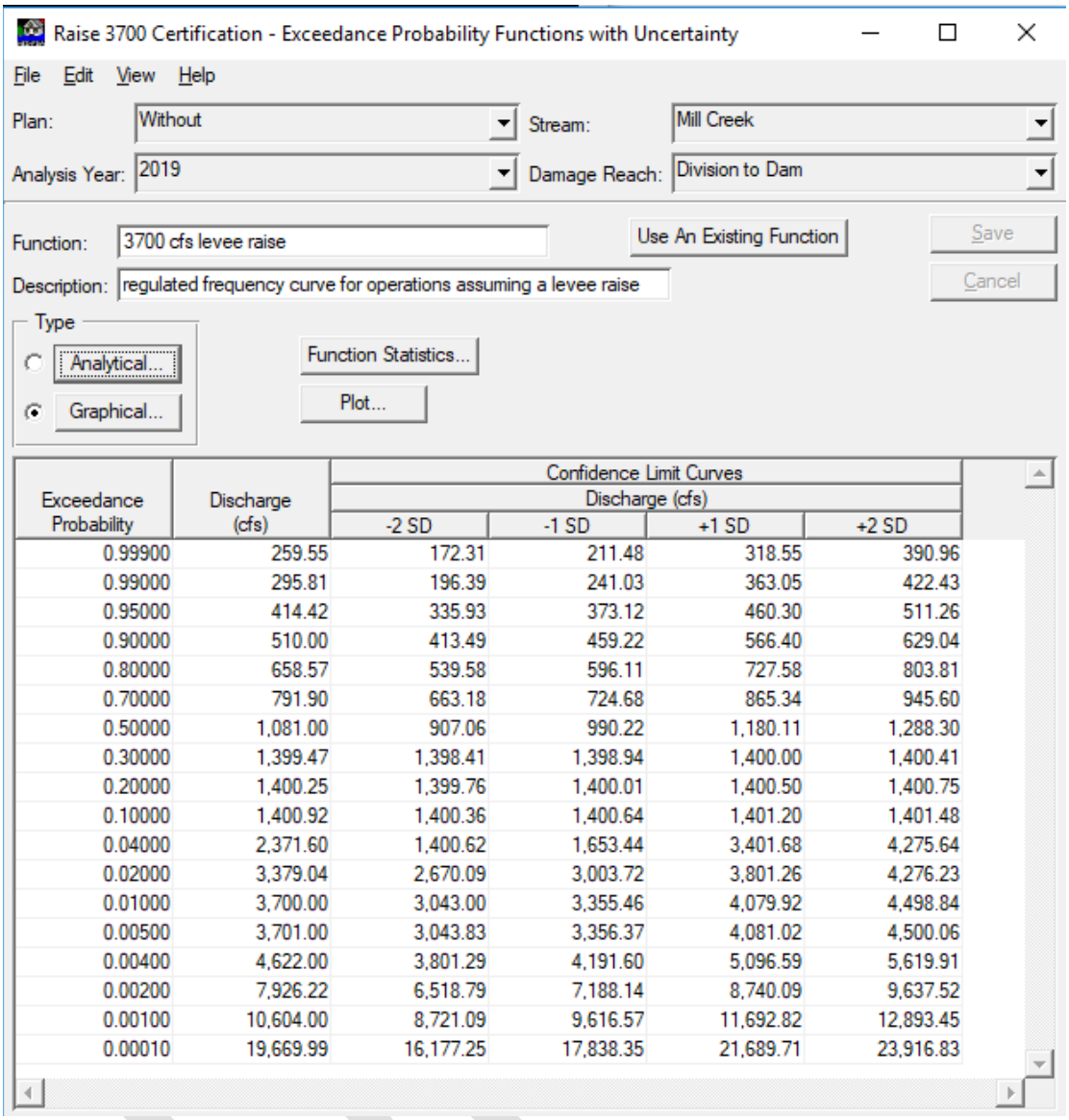
The following data points were input into the model with a period of record of 78 years.



	Exceedance Probability	Discharge (cfs)
1	0.99520	261.00
2	0.98280	327.00
3	0.95360	406.00
4	0.90000	510.00
5	0.60000	927.00
6	0.50000	1081.00
7	0.41040	1249.00
8	0.34710	1399.00
9	0.25000	1400.00
10	0.09130	1401.00
11	0.05000	2069.00
12	0.02194	3336.00
13	0.01000	3700.00
14	0.00500	3701.00
15	0.00400	4622.00
16	0.00334	5634.00
17	0.00236	6494.00
18	0.00226	7272.00
19	0.00170	8875.00
20	0.00144	9371.00
21	0.00100	10604.00
22	0.00018	16960.00
23	0.00010	19670.00

Figure 3-40. Small Levee Raise Regulated Frequency Curve FDA Input

From these points, HEC-FDA generated the following frequency curve table and plot.



Exceedance Probability	Discharge (cfs)	Confidence Limit Curves Discharge (cfs) -2 SD	-1 SD	+1 SD	+2 SD
0.99900	259.55	172.31	211.48	318.55	390.96
0.99000	295.81	196.39	241.03	363.05	422.43
0.95000	414.42	335.93	373.12	460.30	511.26
0.90000	510.00	413.49	459.22	566.40	629.04
0.80000	658.57	539.58	596.11	727.58	803.81
0.70000	791.90	663.18	724.68	865.34	945.60
0.50000	1,081.00	907.06	990.22	1,180.11	1,288.30
0.30000	1,399.47	1,398.41	1,398.94	1,400.00	1,400.41
0.20000	1,400.25	1,399.76	1,400.01	1,400.50	1,400.75
0.10000	1,400.92	1,400.36	1,400.64	1,401.20	1,401.48
0.04000	2,371.60	1,400.62	1,653.44	3,401.68	4,275.64
0.02000	3,379.04	2,670.09	3,003.72	3,801.26	4,276.23
0.01000	3,700.00	3,043.00	3,355.46	4,079.92	4,498.84
0.00500	3,701.00	3,043.83	3,356.37	4,081.02	4,500.06
0.00400	4,622.00	3,801.29	4,191.60	5,096.59	5,619.91
0.00200	7,926.22	6,518.79	7,188.14	8,740.09	9,637.52
0.00100	10,604.00	8,721.09	9,616.57	11,692.82	12,893.45
0.00010	19,669.99	16,177.25	17,838.35	21,689.71	23,916.83

Figure 3-41. Small Levee Raise Regulated Frequency Curve FDA extrapolation

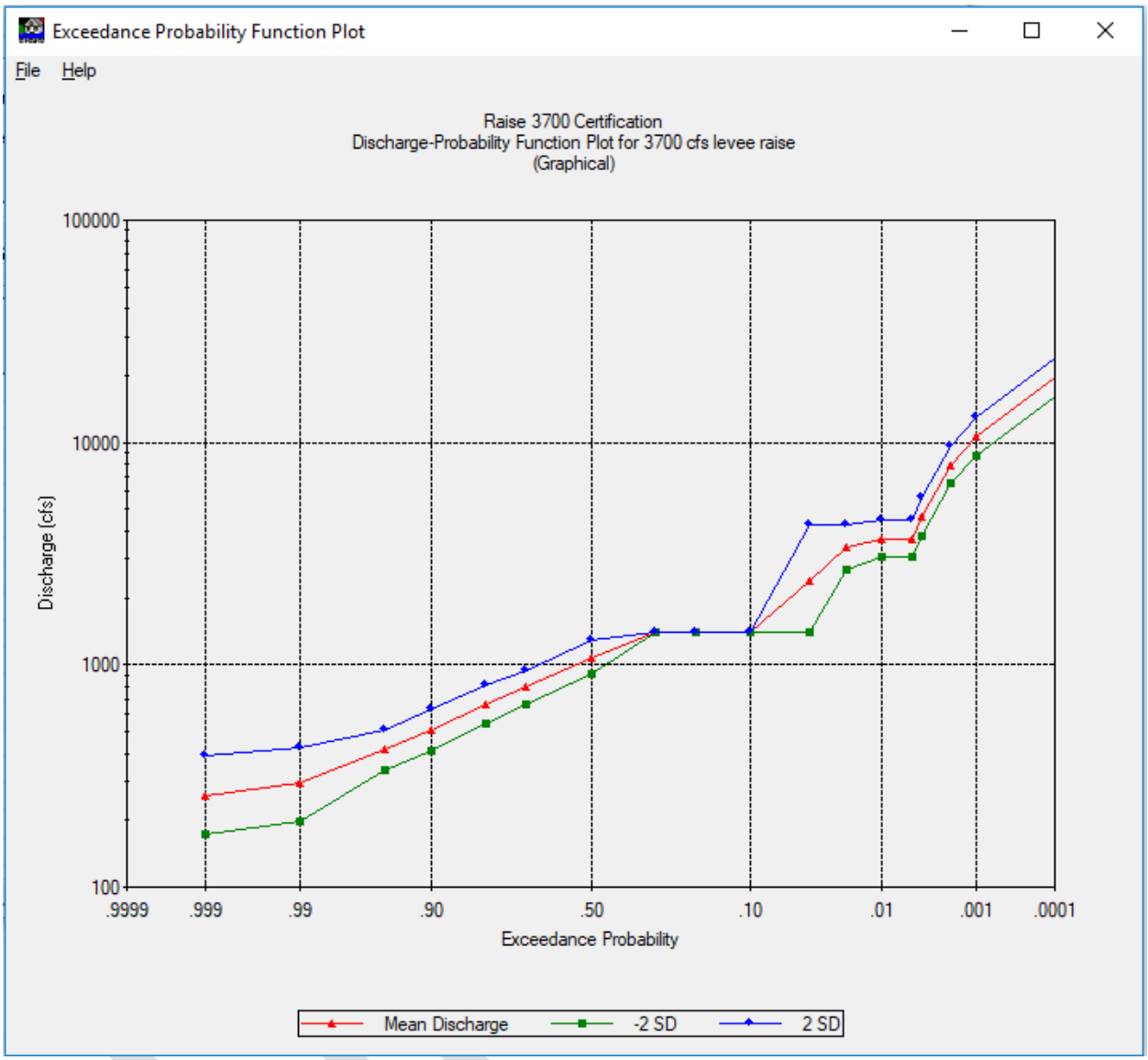


Figure 3-42. Small Levee Raise Regulated Frequency Curve in FDA

Hydraulic Model

For levee sizing the unsteady hydraulic model developed for inundation mapping was adapted to be a steady flow model. The 2D overbank areas were removed and the model was utilized as a steady flow 1D channel only model for levee sizing and certification. Eight steady flow profiles were developed for the for specific percent exceedance flows as shown the following table. These were developed at key points in the regulated frequency curve.

Table 3-17. Small Levee Raise Frequency Flows, Levee Sizing

Probability	Flow (cfs)
0.25	1,400
0.05	2,069
0.02194	3,336
0.01	3,700
0.00404	5,005
0.00232	7,171
0.00184	8,639
0.001	10,604

Channel n values were set to 0.04 for the levee reach.

Evaluation Reaches

The same evaluation reaches were utilized as determined for existing conditions in Table 3-14.

Stage Uncertainty

The same stage uncertainty values, the Standard Deviation of the Error, were utilized as determined for existing conditions in Table 3-15.

Results

To meet levee certification requirements for hydrology and hydraulics outlined in ECB 2019-11 (EC 1165-2-218 Draft), a levee needs a Conditional Non-Exceedance Probability of greater than 0.85 for the 1% event at all index locations.

In order to size the levee raise to meet levee certification standards for the 1-percent chance flood (3,700 cfs), the levee from Myra to 9th Street Bridges will need to be raised 0.7 feet. The following table summarizes the results

Table 3-18. Small Levee Raise Requirements

Critical Index Cross Section	Damage Reach	Conditional Non-Exceedance Probability 1% Exceedance Flood (3,700 cfs)	Levee Raise Required to meet 0.85 Probability for Levee Certification (feet)
12.2216	Division Works to Dam	0.978	0.0
10.9688	Tausick to Division Works	0.885	0.0
10.3597	Wilber to Tausick	0.856	0.0
9.8223	Roosevelt to Wilber	0.975	0.0
7.3084	Myra to 9th	0.737	0.7
5.607	Gose to Myra	0.870	0.0

As the hydraulic model and hydrology are refined a more definitive determination can be made.

3.3.2.3 Moderate Levee Raise Alternative

For this alternative the regulated flow frequency curve was revised to change the regulation target from 3,500 cfs to 4,000 cfs. The regulated frequency curve under this alternative shows the 1-percent chance flood as 4,000 cfs. The rest of this section outline levee sizing to meet levee certification requirements for a 3,700 cfs levee.

<u>Flow Frequencies</u>

The 4,000 cfs Levee Raise Alternative regulated frequency curve was placed into HEC-FDA as a graphical frequency curve. The frequency curve was developed as documented in " Regulated Flow Frequency Curve Development". This frequency curve is unchanged from the existing conditions frequency curve below 3,400 cfs (~0.02 percent flood) and above 10,000 cfs (~ 0.001 percent flood).

The following data points were input into the model with a period of record of 78 years.

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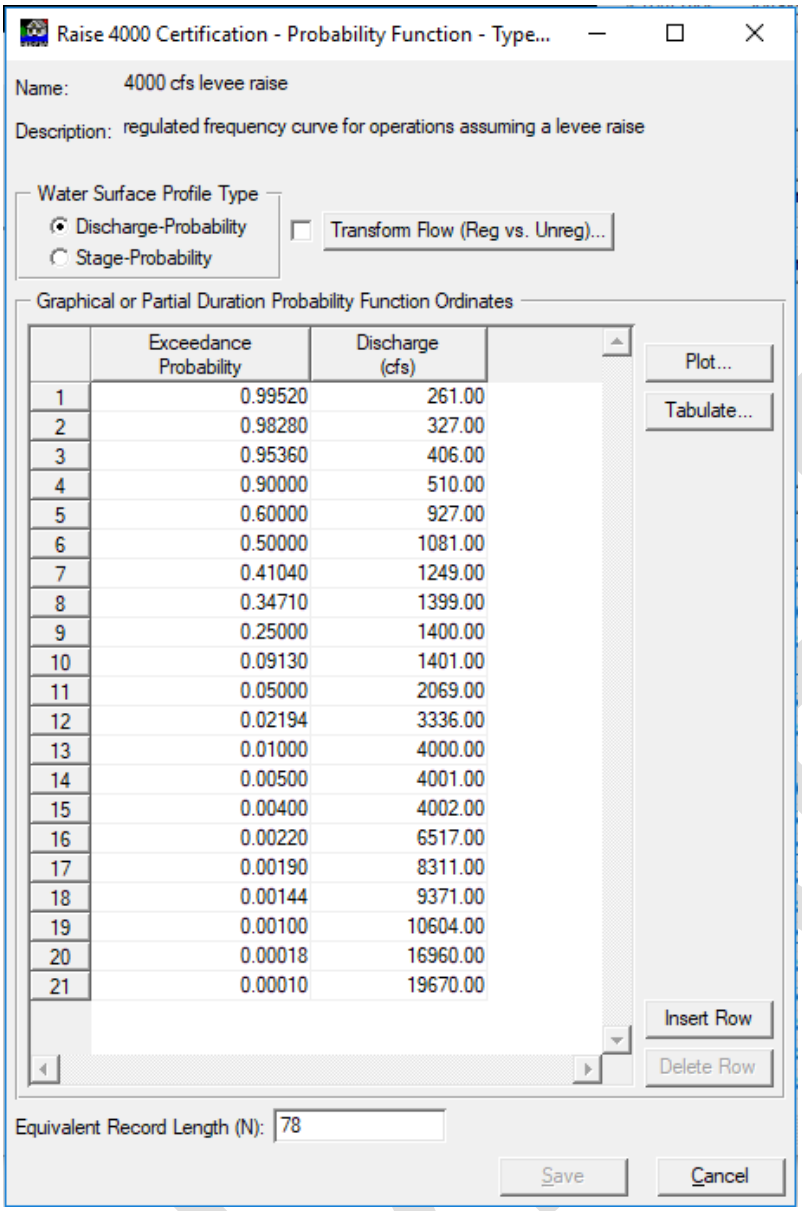
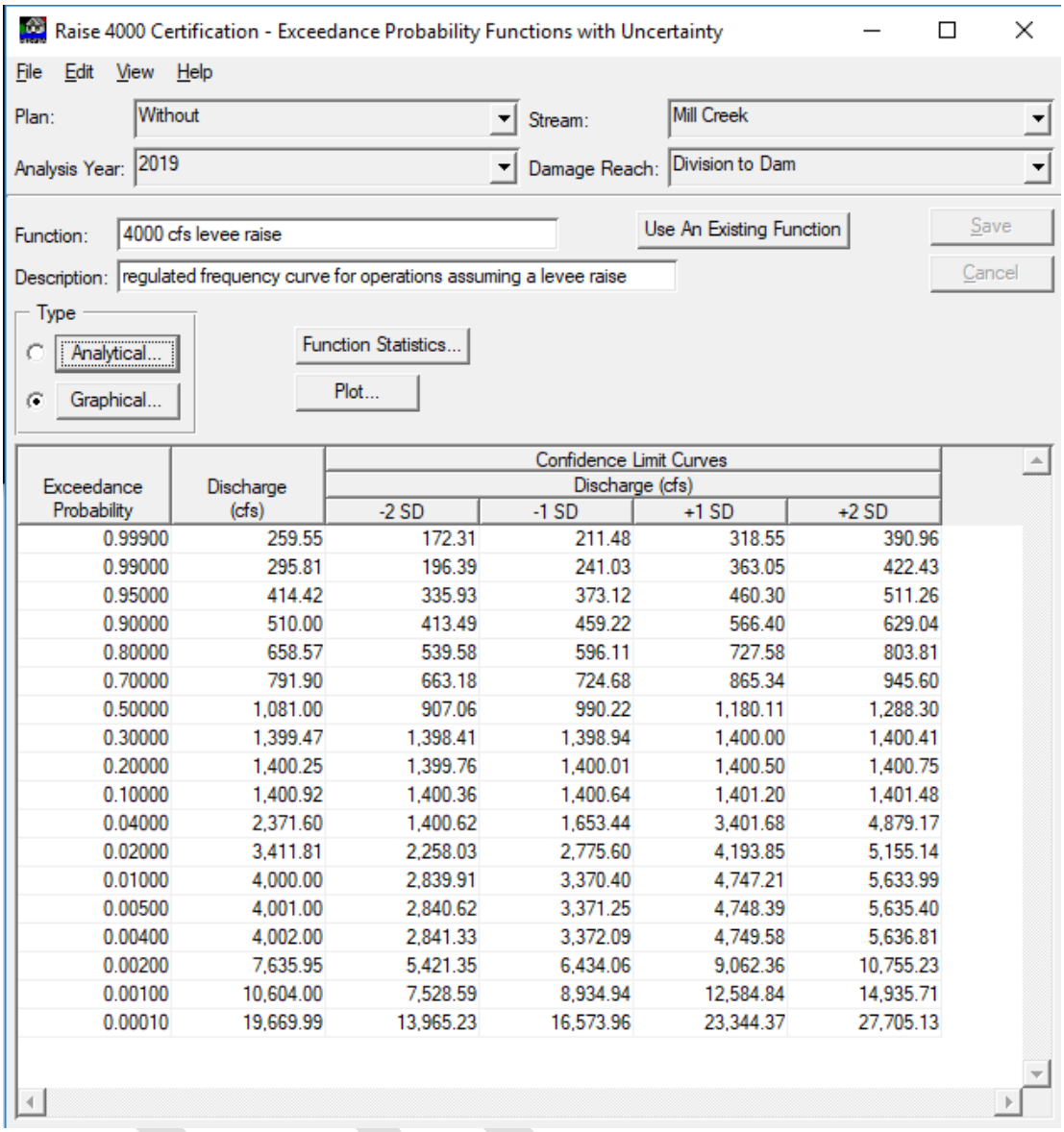


	Exceedance Probability	Discharge (cfs)
1	0.99520	261.00
2	0.98280	327.00
3	0.95360	406.00
4	0.90000	510.00
5	0.60000	927.00
6	0.50000	1081.00
7	0.41040	1249.00
8	0.34710	1399.00
9	0.25000	1400.00
10	0.09130	1401.00
11	0.05000	2069.00
12	0.02194	3336.00
13	0.01000	4000.00
14	0.00500	4001.00
15	0.00400	4002.00
16	0.00220	6517.00
17	0.00190	8311.00
18	0.00144	9371.00
19	0.00100	10604.00
20	0.00018	16960.00
21	0.00010	19670.00

Figure 3-43. Moderate Levee Raise Regulated Frequency Curve FDA Input

From these points, HEC-FDA generated the following frequency curve table and plot.



Exceedance Probability	Discharge (cfs)	Confidence Limit Curves Discharge (cfs) -2 SD	-1 SD	+1 SD	+2 SD
0.99900	259.55	172.31	211.48	318.55	390.96
0.99000	295.81	196.39	241.03	363.05	422.43
0.95000	414.42	335.93	373.12	460.30	511.26
0.90000	510.00	413.49	459.22	566.40	629.04
0.80000	658.57	539.58	596.11	727.58	803.81
0.70000	791.90	663.18	724.68	865.34	945.60
0.50000	1,081.00	907.06	990.22	1,180.11	1,288.30
0.30000	1,399.47	1,398.41	1,398.94	1,400.00	1,400.41
0.20000	1,400.25	1,399.76	1,400.01	1,400.50	1,400.75
0.10000	1,400.92	1,400.36	1,400.64	1,401.20	1,401.48
0.04000	2,371.60	1,400.62	1,653.44	3,401.68	4,879.17
0.02000	3,411.81	2,258.03	2,775.60	4,193.85	5,155.14
0.01000	4,000.00	2,839.91	3,370.40	4,747.21	5,633.99
0.00500	4,001.00	2,840.62	3,371.25	4,748.39	5,635.40
0.00400	4,002.00	2,841.33	3,372.09	4,749.58	5,636.81
0.00200	7,635.95	5,421.35	6,434.06	9,062.36	10,755.23
0.00100	10,604.00	7,528.59	8,934.94	12,584.84	14,935.71
0.00010	19,669.99	13,965.23	16,573.96	23,344.37	27,705.13

Figure 3-44. Moderate Levee Raise Regulated Frequency Curve FDA Extrapolation

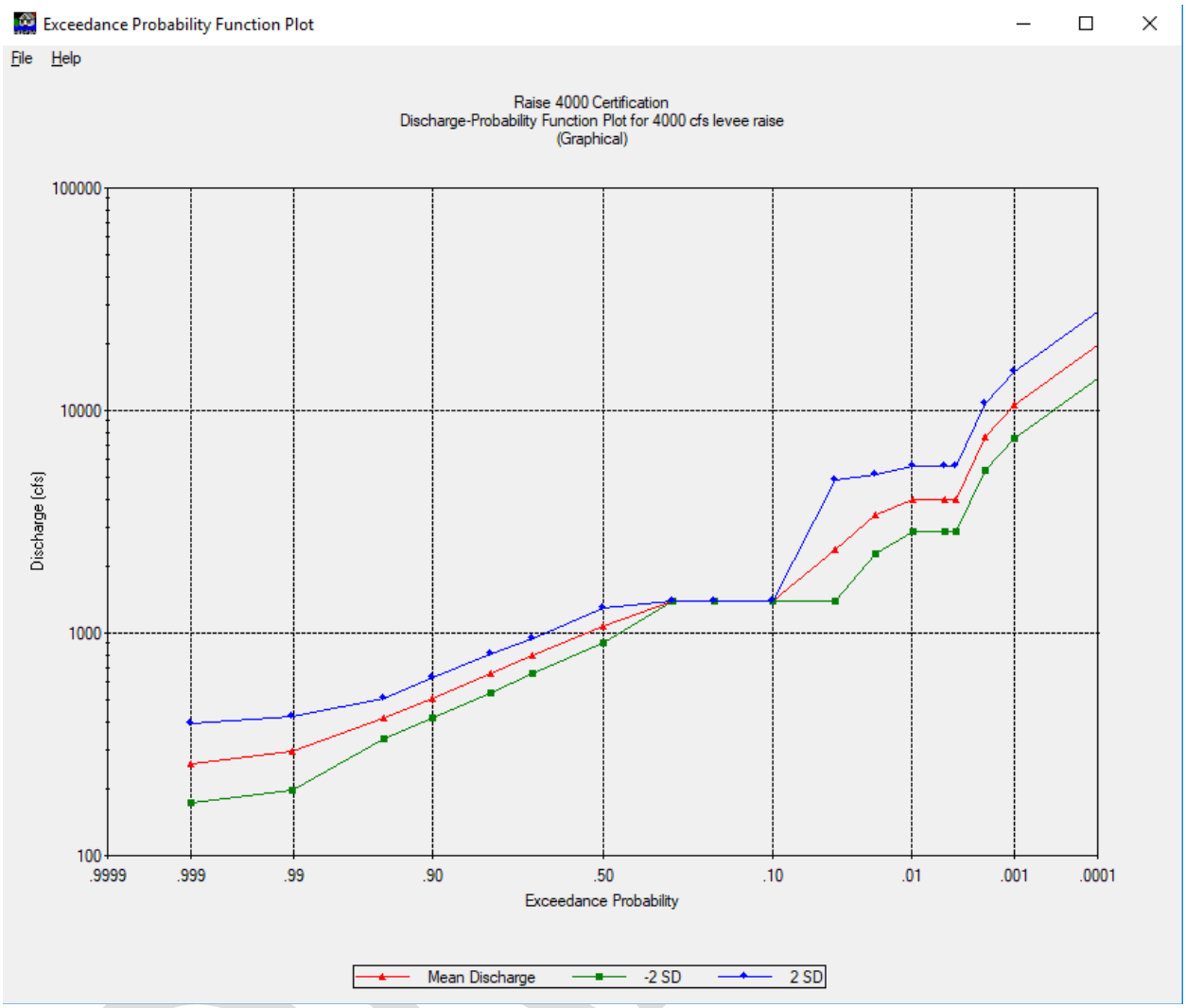


Figure 3-45. Moderate Levee Raise Regulated Frequency Curve in FDA

Hydraulic Model

For levee sizing the unsteady hydraulic model developed for inundation mapping was adapted to be a steady flow model. The 2D overbank areas were removed and the model was utilized as a steady flow 1D channel only model for levee sizing and certification. Eight steady flow profiles were developed for the for specific percent exceedance flows as shown the following table. These were developed at key points in the regulated frequency curve.

Table 3-19. Moderate Levee Raise Frequency Flows, Levee Sizing

Probability	Flow (cfs)
0.25	1,400
0.05	2,069
0.02194	3,336
0.01	4,000
0.00404	5,005
0.00232	7,171
0.00184	8,639
0.001	10,604

Channel n values were set to 0.04 for the levee reach.

Evaluation Reaches

The same evaluation reaches were utilized as determined for existing conditions in Table 3-14.

Stage Uncertainty

The same stage uncertainty values, the Standard Deviation of the Error, were utilized as determined for existing conditions in Table 3-15.

Results

To meet levee certification requirements for hydrology and hydraulics outlined in ECB 2019-11 (EC 1165-2-218 Draft), a levee needs a Conditional Non-Exceedance Probability of greater than 0.85 for the 1% event at all index locations.

In order to size the levee raise to meet levee certification standards for the 1-percent chance flood (4,000 cfs), the levee will need to be raised in multiple locations. The following table summarizes the results

Table 3-20. Moderate Levee Raise Requirements

Critical Index Cross Section	Damage Reach	Conditional Non-Exceedance Probability 1% Exceedance Flood (4,000 cfs)	Levee Raise Required to meet 0.85 Probability for Levee Certification (feet)
12.2216	Division Works to Dam	0.969	0.0
10.9688	Tausick to Division Works	0.811	0.3
10.3597	Wilber to Tausick	0.823	0.2
9.8223	Roosevelt to Wilber	0.968	0.0
7.3084	Myra to 9th	0.563	1.5
5.607	Gose to Myra	0.831	0.2

As the hydraulic model and hydrology are refined a more definitive determination can be made.

3.3.3 Inundation Mapping and FDA Input

Existing Conditions

The Existing Conditions scenario models Mill Creek as it appears today. No levee raises or excavations are modeled under this scenario.

Table 3-21. Flows for Existing Conditions Scenario

Existing Conditions									
Return Period (yr):	1000	694	543	431	248	56 to 176	46	20	11
Probability:	0.001	0.00144	0.00184	0.00232	0.00404	0.01758 to 0.00568	0.02194	0.05	0.0913
Peak Flow (cfs):	10604	9371	8640	7171	5005	3500	3337	2069	1400

Each of these flows were modeled and 2D overbank inundation maps were generated. This was completed for the following 5 scenarios:

1. No Failure
2. Levee Breach upstream Yellowhawk Division Structure
3. Levee Breach upstream Tausik Way Bridge
4. Channel Blockage upstream Otis Street Bridge
5. Channel Blockage in Tunnel exiting waters upstream Colville Street.

Small Excavation Alternative

Under the Small Excavation scenario a small amount of Mill Creek Reservoir bottom would be removed. This excavation would increase the reservoir's capacity and would allow for a little more flexibility of regulating the reservoir during a storm event.

Table 3-22. Flows for Small Excavation Scenario

Small Excavation									
Return Period (yr):	1000	694	538	442		57 to 176	46	20	11
Probability:	0.001	0.00144	0.00186	0.00226		0.01758 to 0.00506	0.02194	0.05	0.0913
Peak Flow (cfs):	10544	9371	8476	6803		3500	3337	2074	1400

Each of these flows were modeled and 2D overbank inundation maps were generated. This was completed for the following 5 scenarios:

1. No Failure
2. Levee Breach upstream Yellowhawk Division Structure
3. Levee Breach upstream Tausik Way Bridge
4. Channel Blockage upstream Otis Street Bridge
5. Channel Blockage in Tunnel exiting waters upstream Colville Street.

Moderate Excavation Alternative

Under the Moderate Excavation scenario more or the Mill Creek Reservoir bottom would be removed when compared to the Small Excavation scenario. This Moderate Excavation would increase the reservoir's capacity and would allow for a more flexibility when regulating the reservoir during a storm event.

Table 3-23. Flows for Moderate Excavation Scenario

Moderate Excavation									
Return Period (yr):	1000	704		450	262	57 to 251	46	20	11
Probability:	0.001	0.00142		0.00222	0.00382	0.01758 to 0.00398	0.02194	0.05	0.0913
Peak Flow (cfs):	10311	9265		6162	4165	3500	3340	2088	1400

Each of these flows were modeled and 2D overbank inundation maps were generated. This was completed for the following 5 scenarios:

1. No Failure
2. Levee Breach upstream Yellowhawk Division Structure
3. Levee Breach upstream Tausik Way Bridge
4. Channel Blockage upstream Otis Street Bridge
5. Channel Blockage in Tunnel exiting waters upstream Colville Street.

Small Levee Raise Alternative

The Small Levee Raise scenario would raise the Mill Creek levees a small amount. This levee raise would be at current low spots increase the levees' ability to handle a more flow.

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Table 3-24. Flows for Small Levee Raise Scenario

Small Levee Raise									
Return Period (yr):	1000	694	588	442	424	65 to 200	46	20	11
Probability:	0.001	0.00144	0.0017	0.00226	0.00236	0.0153 to 0.005	0.02194	0.05	0.0913
Peak Flow (cfs):	10604	9371	8875	7273	6494	3700	3337	2069	1400

Each of these flows were modeled and 2D overbank inundation maps were generated. This was completed for the following 5 scenerios:

1. No Failure
2. Levee Breach upstream Yellowhawk Division Structure
3. Levee Breach upstream Tausik Way Bridge
4. Channel Blockage upstream Otis Street Bridge
5. Channel Blockage in Tunnel exiting waters upstream Colville Street.

<u>Moderate Levee Raise Alternative</u>

The Moderate Levee Raise scenario would raise the Mill Creek levees more than the Small Levee Raise scenario. This levee raise would be at current low spots increase the levees' ability to handle a more flow.

Table 3-25. Flows for Moderate Levee Raise Scenario

Moderate Levee Raise									
Return Period (yr):	1000	694	526	455		76 to 250	46	20	11
Probability:	0.001	0.00144	0.0019	0.0022		0.0132 to 0.004	0.02194	0.05	0.0913
Peak Flow (cfs):	10604	9371	8311	6517		4000	3337	2069	1400

Each of these flows were modeled and 2D overbank inundation maps were generated. This was completed for the following 5 scenerios:

1. No Failure
2. Levee Breach upstream Yellowhawk Division Structure
3. Levee Breach upstream Tausik Way Bridge
4. Channel Blockage upstream Otis Street Bridge
5. Channel Blockage in Tunnel exiting waters upstream Colville Street.

<u>Small Excavation and Small Levee Raise Alternative</u>

The Small Levee Raise and Small Levee Raise scenario would raise the Mill Creek levees a small amount and also excavate a small amount of Mill Creek Reservoir bottom. This scenario would

increase the reservoir's capacity and also raise the levees at low spots. Doing both the Small Levee Raise and the Small Excavation and would allow for more flow capacity and more flexibility when regulating the reservoir during a storm event.

Table 3-26. Flows for Small Levee Raise and Small Levee Raise Scenario

Small Excav. + Small Levee Raise									
Return Period (yr):	1000	694	581	439	244	65 to 239	46	20	11
Probability:	0.001	0.00144	0.00172	0.00228	0.0041	0.0153 to 0.00418	0.02194	0.05	0.0913
Peak Flow (cfs):	10544	9342	8778	6432	4225	3700	3337	2074	1400

Each of these flows were modeled and 2D overbank inundation maps were generated. This was completed for the following 5 scenerios:

1. No Failure
2. Levee Breach upstream Yellowhawk Division Structure
3. Levee Breach upstream Tausik Way Bridge
4. Channel Blockage upstream Otis Street Bridge
5. Channel Blockage in Tunnel exiting waters upstream Colville Street.

<u>FDA Input</u>

The output from the hydraulic analysis was provided for HEC-FDA use to determine economics. Locations for Functions coincide with Fragility Curve locations. These were determined as locations in the design channel most likely to fail based on a PFMA analysis described in the Engineering Appendix.

The Standard Deviation of the Error was determined by computation of uncertainty in friction and wave height. The manning's n values were varied in the HEC-RAS model to determine the standard deviation for the index locations for each flow frequency. The low channel manning's n was set to 0.03 for the levee reach, and 0.01 for the concrete channel. The high manning's n was set to 0.05 for the levee reach and 0.02 for the concrete channel. The best estimate being 0.04 for the levee reach and 0.015 for the concrete channel. The high stage was further adjusted by 1.5 foot for wave action as observed at stabilizers near the levee during the 1996 flood. The following equation was used to determine the Standard deviation of error for each flows surrounding the design target at index cross sections.

Standard Deviation of the Error $= [(\text{high stage} - \text{expected stage})^2 + (\text{low stage} - \text{expected stage})^2 / 2]^{1/2}$

Next, the equation results were compared to Table 5-2 in EM 1110-2-1619 to assure that these values met minimum standard deviation of error for stage as recommended in the EM. For

these reaches the table indicated something between 0.7 and 1.3 would be a minimum value. Finally, to get smoothed results a single value was used for all flood frequencies at each index location based on evaluation of the calculated Standard Deviation of the Error and the EM guidance. This value was selected from flows between 3,500 cfs and 5,000 cfs as this is the critical flow range for levee sizing. The reason for selecting a single value was based on lower confidence in the values obtained from the rough hydraulic model where instabilities have not all been worked out, and meeting the minimum range given in the EM. Model inputs are shown in the following table.

Table 3-27. Stage-Discharge Function with Uncertainty

FDA Menu: Stage-Discharge Function with Uncertainty					
Function	**RAS Location**	**Physical Description**	**Distribution Type**	**Stage Where Error Becomes Constant**	**Standard Deviation of Error for Entered Stage**
8.183	8.183	Upstream Colville St. Bridge (tunnel)	Normal	956.12	1.1
8.69	8.69	Upstream Otis St. Bridge	Normal	984.39	1.1
10.9688	10.9688	Upstream Tausick Bridge	Normal	1140.74	1.3
11.5168	11.5168	Upstream Division Works	Normal	1177.27	1.1

Levee Features are shown in the following table for the following alternatives:

1. Existing Conditions
2. Small Excavation,
3. Moderate Excavation,
4. Small Levee Raise
5. Small Excavation and Small Levee Raise

Table 3-28. Levee Features Most Alternatives

FDA Menu: Levee Features	
Levee Name (cross section Location)	**Top of Levee Stage (feet NAVD)**
8.183	963.87
8.69	989.81
10.9688	1144.89
11.5168	1182.92

Levee Features are shown in the following table for the Moderate Levee Raise Alternative.

Table 3-29. Levee Features for the Moderate Levee Raise Alternative

FDA Menu: Levee Features	
Levee Name (cross section Location)	**Top of Levee Stage (feet NAVD)**
8.183	964.17
8.69	990.11
10.9688	1145.19
11.5168	1183.22

3.4 MODEL SUMMARY RESULTS

Information in this section is to summarize results and compare alternatives. The following figure present the mean Annual Exceedance Probability (AEP) and Long Term Exceedance Probability (LTEP) computed for each alternative. The LTEP is the likelihood of exceedance at least once in the specified period and is computed as $1-(1-AEP)^N$, where N = number of years.

Table 3-30. Levee Risk Assessment for Division Works to Diversion Dam (12.2216)

Alternative	Conditional Non-Exceedance Probability of Levee Overtopping for 1% Flood	Mean (Expected) AEP for Levee Overtopping	LTEP (Probability of Exceedance over Indicated Time)		
			10 Years	30 Years	50 Years
Existing Conditions	0.9827	0.002	0.0203	0.0596	0.0973
Small Excavation	0.9827	0.002	0.0195	0.0575	0.094
Moderate Excavation	0.9827	0.0019	0.0187	0.0551	0.0901
Small Levee Raise	0.9783	0.0021	0.0207	0.0609	0.0994
Moderate Levee Raise	0.969	0.0022	0.0217	0.0636	0.1038
Small Excavation and Small Levee Raise	0.9783	0.002	0.0201	0.0592	0.0966
Operational Change – Increased Initial Regulated Flow	0.982	0.0024	0.0238	0.0697	0.1134

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Table 3-31. Levee Risk Assessment for Division Works to Diversion Dam (10.9688)

Alternative	Conditional Non-Exceedance Probability of Levee Overtopping for 1% Flood	Mean (Expected) AEP for Levee Overtopping	LTEP (Probability of Exceedance over Indicated Time)		
			10 Years	30 Years	50 Years
Existing Conditions	0.9178	0.0064	0.0621	0.1751	0.2744
Small Excavation	0.9178	0.006	0.0582	0.1647	0.2591
Moderate Excavation	0.9178	0.0056	0.0547	0.1554	0.2453
Small Levee Raise	0.8849	0.0068	0.0658	0.1846	0.2883
Moderate Levee Raise	0.86	0.0063	0.0614	0.1731	0.2715
Small Excavation and Small Levee Raise	0.8849	0.0065	0.0635	0.1787	0.2796
Operational Change – Increased Initial Regulated Flow	0.9123	0.0069	0.0673	0.1885	0.294

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Table 3-32. Levee Risk Assessment for Division Works to Diversion Dam (10.3597)

Alternative	Conditional Non-Exceedance Probability of Levee Overtopping for 1% Flood	Mean (Expected) AEP for Levee Overtopping	LTEP (Probability of Exceedance over Indicated Time)		
			10 Years	30 Years	50 Years
Existing Conditions	0.8727	0.0185	0.1703	0.4289	0.6069
Small Excavation	0.8727	0.0183	0.1687	0.4255	0.603
Moderate Excavation	0.8727	0.0181	0.1671	0.4221	0.5991
Small Levee Raise	0.8559	0.0187	0.1717	0.4316	0.61
Moderate Levee Raise	0.8574	0.0143	0.1343	0.3513	0.5138
Small Excavation and Small Levee Raise	0.8559	0.0185	0.1706	0.4294	0.6074
Operational Change – Increased Initial Regulated Flow	0.8695	0.0232	0.2092	0.5055	0.6908

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Table 3-33. Levee Risk Assessment for Division Works to Diversion Dam (9.8223)

Alternative	Conditional Non-Exceedance Probability of Levee Overtopping for 1% Flood	Mean (Expected) AEP for Levee Overtopping	LTEP (Probability of Exceedance over Indicated Time)		
			10 Years	30 Years	50 Years
Existing Conditions	0.979	0.0025	0.0248	0.0727	0.1182
Small Excavation	0.979	0.0025	0.0243	0.0712	0.1159
Moderate Excavation	0.979	0.0024	0.0237	0.0693	0.1129
Small Levee Raise	0.9747	0.0026	0.0253	0.074	0.1203
Moderate Levee Raise	0.9687	0.0026	0.0259	0.0757	0.1229
Small Excavation and Small Levee Raise	0.9747	0.0025	0.0249	0.0729	0.1185
Operational Change – Increased Initial Regulated Flow	0.9783	0.0033	0.0328	0.0953	0.1537

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Table 3-34. Levee Risk Assessment for Division Works to Diversion Dam (7.3084)

Alternative	Conditional Non-Exceedance Probability of Levee Overtopping for 1% Flood	Mean (Expected) AEP for Levee Overtopping	LTEP (Probability of Exceedance over Indicated Time)		
			10 Years	30 Years	50 Years
Existing Conditions	0.8418	0.0085	0.0821	0.2267	0.3485
Small Excavation	0.8417	0.0078	0.0756	0.21	0.3249
Moderate Excavation	0.8417	0.0074	0.072	0.2008	0.3117
Small Levee Raise	0.8773	0.0065	0.0632	0.178	0.2787
Moderate Levee Raise	0.8497	0.0057	0.0554	0.1573	0.2481
Small Excavation and Small Levee Raise	0.8773	0.0061	0.0593	0.1676	0.2634
Operational Change – Increased Initial Regulated Flow	0.8099	0.0073	0.0709	0.198	0.3077

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Table 3-35. Levee Risk Assessment for Division Works to Diversion Dam (5.607)

Alternative	Conditional Non-Exceedance Probability of Levee Overtopping for 1% Flood	Mean (Expected) AEP for Levee Overtopping	LTEP (Probability of Exceedance over Indicated Time)		
			10 Years	30 Years	50 Years
Existing Conditions	0.8911	0.0097	0.0925	0.2527	0.3847
Small Excavation	0.8911	0.0094	0.0903	0.2472	0.3771
Moderate Excavation	0.8911	0.0092	0.0882	0.242	0.3699
Small Levee Raise	0.8696	0.0099	0.095	0.2589	0.3931
Moderate Levee Raise	0.8572	0.0086	0.0827	0.2282	0.3506
Small Excavation and Small Levee Raise	0.8696	0.0098	0.0936	0.2554	0.3884
Operational Change – Increased Initial Regulated Flow	0.8882	0.0127	0.1204	0.3196	0.4736

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SECTION 4 - TENATIVELY SELECTED PLAN OUTCOME

Note: The following conclusions were not drawn solely from hydrology and hydraulics analysis but from the project development team efforts as a whole. They are included here to inform the reader of the study process that led to the next hydrology and hydraulics section.

The following alternatives were included in the full modeled analysis. These are:

- Rehab of the concrete channel
- Small excavation
- Small levee raise
- Moderate excavation
- Moderate levee raise
- Small excavation and small levee raise
- Operational Change

Following analysis from the product delivery team, the Tentatively Selected Plan (TSP) is an **operations change and a levee raise**.

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SECTION 5 - INTERIM ANALYSIS FOR THE TSP

Following analysis from the product delivery team, the Tentatively Selected Plan (TSP) is an operations change and a levee raise. While these two alternatives were individually modeled by Hydraulics and Hydrology Branch prior to the TSP decision, the combinations of these two selected plans was not modeled. This interim analysis completes the modeling for this alternative.

During the intervening time from the TSP meeting till this juncture, models were enhanced – particularly the FDA model and the Hydraulic Model. This interim analysis relies on these updated models. As a result, the alternative results of this section are not directly comparable to any prior work in this report. This work included updated existing conditions and a single rendering of the TSP alternative.

5.1 OVERVIEW AND ALTERNATIVE DESCRIPTION

This effort used the Unregulated Flow Frequency Curves and Existing Regulated Flow Frequency Curves from Section 3.0 Full Modeled Alternatives. Following the same methodology new Regulated Flow Frequency Curves were developed for the alternative Small Levee Raise with Operational Change. The resulting regulated peak frequencies were utilized in an unsteady flow HEC-RAS 1D/2D model to generate inundation mapping sets for existing conditions and the alternative. The evaluation is shown in Table 5-1.

Table 5-1. Interim TSP Analysis

Project Alternative	Project Flow Objective (cubic feet per second)	Flood Risk Target in Design Channel For Flow Objective (Percent AEP)
Existing Conditions	3,500 cfs through levees	0.57-Percent (175 Year)
Small Levee Raise and Operational Change – Increased Initial Regulated Flow to 1,700 cfs	3,700 cfs through levees	0.458-Percent (218 Year)

The following are the alternative descriptions:

<u>Existing Conditions</u>

This is the base alternative. This alternative utilizes all existing facilities and operations plans that are present today. An unregulated inflow Frequency curve was developed in HEC-SSP and used for this and all other project alternatives. The HEC-WAT model was utilized to randomly sample unregulated inflow events and route them through the HEC-ResSim model to create regulated outflows. The HEC-ResSim model is an enhanced version of the DSMS model. The elevation-storage table used in the DSMS model was based on a survey conducted in 1983, so a

new elevation-storage table was produced using a LiDAR terrain set from 2011. A regulated outflow frequency curve was developed.

Eight hydrographs, representing specific regulated flow frequencies were routed through the RAS model to create inundation areas. The RAS model was an enhanced version of the model used in Section 3.0 Fully Modeled Alternatives. The model now includes more detail in the reach from Colville Street to Third Avenue and improved ineffective areas to better stability. In addition, better calibration was developed for flow through bridges in the concrete channel. The model was developed to run with 5 scenarios. These included:

1. Channel functions as designed.
2. A 90% blockage of the tunnel portion of the concrete channel due to building collapse
3. A 50% blockage of the Otis Street Bridge due to upstream wall failure
4. Overtopping failure of the levee upstream the Tausick Way Bridge
5. Overtopping failure of the levee upstream the Yellowhawk/Garrison Division works.

The resulting inundation areas were provided to the economist for use in HEC-FDA.

<u>Small Levee Raise and Operations Change</u>

For this alternative the maximum design flow for the project was increased from 3,500 cfs to 3,700 cfs. And the initial regulated flow was increased from 1,400 cfs to 1,700 cfs. The design flow also coincides with the 1-percent chance flood. The levees were raised such that they could pass a 3,700 cfs 1 percent chance flood and be certifiable for the National Flood Insurance Program. The criteria was from ECB 2019-11 (EC 1165-2-218 Draft).

The WAT model was utilized to randomly sample unregulated inflow events and route them through the HEC-ResSim model to create regulated outflows. The table referenced by the diversion rule was changed to increase the maximum regulated flow from 3,500 cfs to 3,700 cfs in the HEC-ResSim model. A regulated outflow frequency curve was developed.

Eight hydrographs, representing specific regulated flow frequencies were routed through the RAS model to create inundation areas. The RAS model was the same as the existing conditions model except for the areas where the levees were raised. Levees were raised in this model to pass 3,700 cfs with 85-percent conditional non-exceedance probability. No changes to the concrete channel were evaluated as its design was for flows in excess of this amount. The model was developed to run with 5 scenarios. These included:

1. Channel functions as designed.
2. A 90% blockage of the tunnel portion of the concrete channel due to building collapse
3. A 50% blockage of the Otis Street Bridge due to upstream wall failure
4. Overtopping failure of the levee upstream the Tausick Way Bridge
5. Overtopping failure of the levee upstream the Yellowhawk/Garrison Division works.

The resulting inundation areas were provided to the economist for use in HEC-FDA.

5.2 REGULATED FLOW FREQUENCY CURVE DEVELOPMENT

The regulated flow frequency curves for each alternative were developed by regulating 50,000 randomly sampled unregulated hydrographs. Each regulated frequency curve is a plot of the 50,000 regulated peaks at their corresponding probabilities, using Weibull plotting positions (Figure 3-27). The random sampling was done using the HEC-WAT program and the hydrographs were regulated using HEC-ResSim, which was run via the HEC-WAT.

5.2.1 Unregulated Frequency Curve Development

The unregulated frequency curve was the same as used in SECTION 3 - Full Modeled Alternatives

5.2.2 HEC-ResSim Model Development

5.2.2.1 Existing Conditions

The existing conditions model was the same as used in SECTION 3 - Full Modeled Alternatives

5.2.2.2 Combined Small Levee Raise and Operations Change

The project operations were modified such that flow is diverted to the reservoir only after Mill Creek flows exceed 1,700 cfs. In addition, the levees were raised to accommodate 3,700 cfs. These changes were modeled by altering the Flood Control Rule Curve inflection points, as describe in sections 3.2.2.3 and 3.2.2.6.

5.2.3 Regulated Frequency Curves for Existing Conditions and Alternative

The output from the HEC-WAT model is the regulated flow frequency curves. These are presented for existing conditions and for the alternative as well as how they are input in HEC-FDA as follows:

<u>Existing Conditions</u>

The unregulated and regulated flow frequency curve from HEC-WAT output is shown in Figure 5-1.

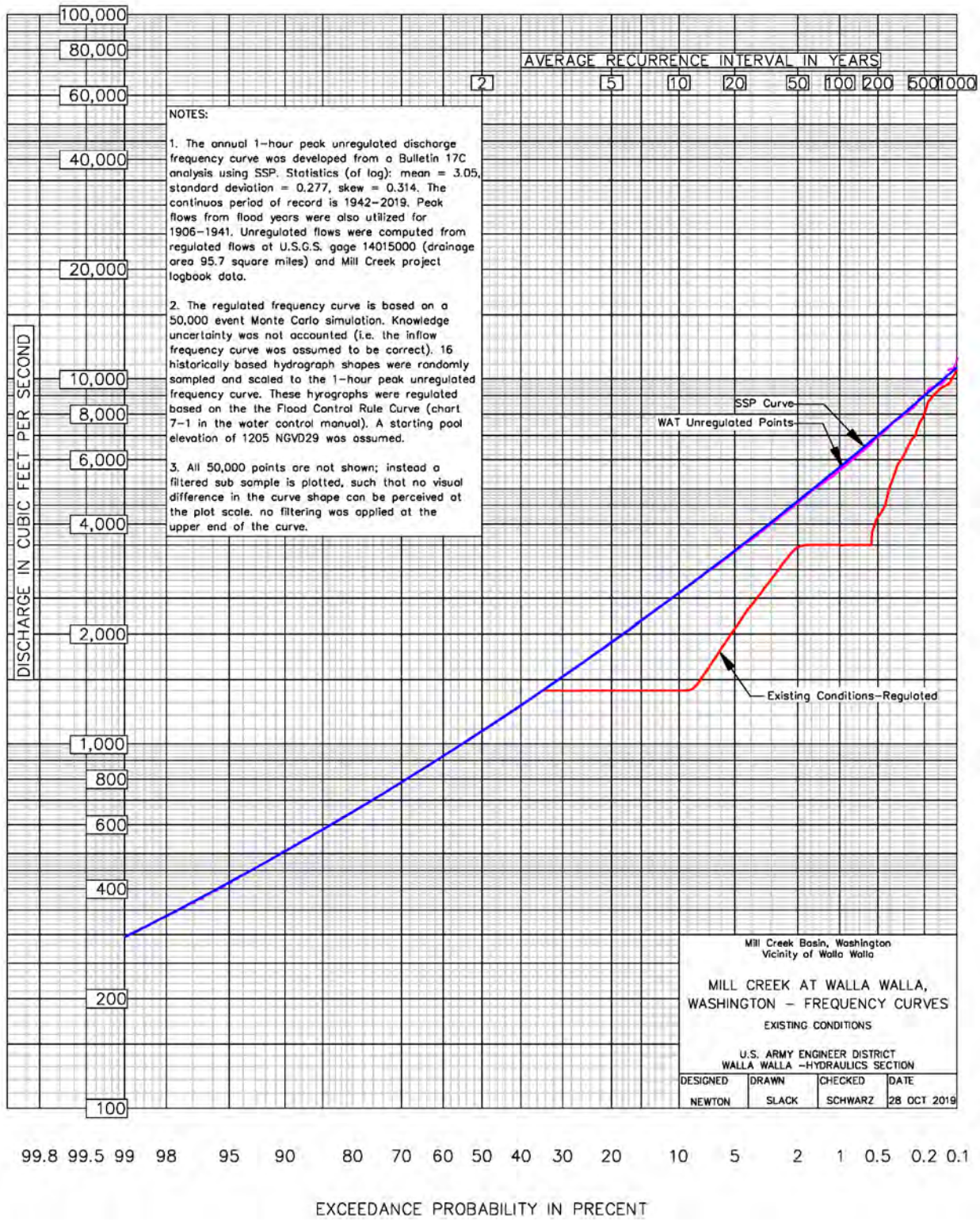


Figure 5-1. Existing Conditions Regulated Flow Frequency Curve

This operation maintains flows to the channel flow target of 3,500 cfs up to the 0.568-percent chance flood (175 Year).

The unregulated frequency curve was entered into HEC-FDA as an Analytical Type with Log Pearson III Statistics: Mean of 3.05, Standard Deviation of 0.2777 and skew of 0.314 with an equivalent record length of 78 years.

The conversion of unregulated to regulation was entered as Transform with a Triangular Distribution Type, see following Figure. This was based on a scatter point plot (inflow peak verses outflow peak) from HEC-WAT output.

Existing conditions - Transform Flow (Regulated vs. Unregulated)

File Edit View Help

Distribution Type: None / Normal / Triangular / Log Normal

	Inflow (cfs)	Outflow (cfs)	Minimum Outflow (cfs)	Maximum Outflow (cfs)
1	207.0	207.0	207.000	207.000
2	295.0	295.0	295.000	295.000
3	417.0	417.0	417.000	417.000
4	508.0	508.0	508.000	508.000
5	651.0	651.0	651.000	651.000
6	785.0	785.0	785.000	785.000
7	1085.0	1085.0	1085.000	1085.000
8	1439.0	1400.0	1400.000	1400.000
9	2743.0	1401.0	1401.000	1401.000
10	3481.0	2069.0	2069.000	2500.000
11	4884.0	3500.0	3500.000	4000.000
12	6965.0	3501.0	3501.000	6000.000
13	7438.0	5005.0	3502.000	7000.000
14	8734.0	7171.0	3503.000	8700.000
15	9260.0	8639.0	4500.000	9200.000
16	10741.0	10741.0	10000.000	10741.000
17	18631.0	18631.0	18631.000	18631.000
18				
19				

Plot... Tabulate... Save Cancel

Figure 5-2. Existing Conditions Unregulated to Regulated Transform

This approach differs from the approach in SECTION 3 - Full Modeled Alternatives. In an effort to better define uncertainty, entering the unregulated frequency with uncertainty and the conversion to regulated with uncertainty, results in a more accurate combined uncertainty.

<u>Small Levee Raise with Operations Change</u>

The unregulated and regulated flow frequency curve from HEC-WAT output is shown in Figure 5-3.

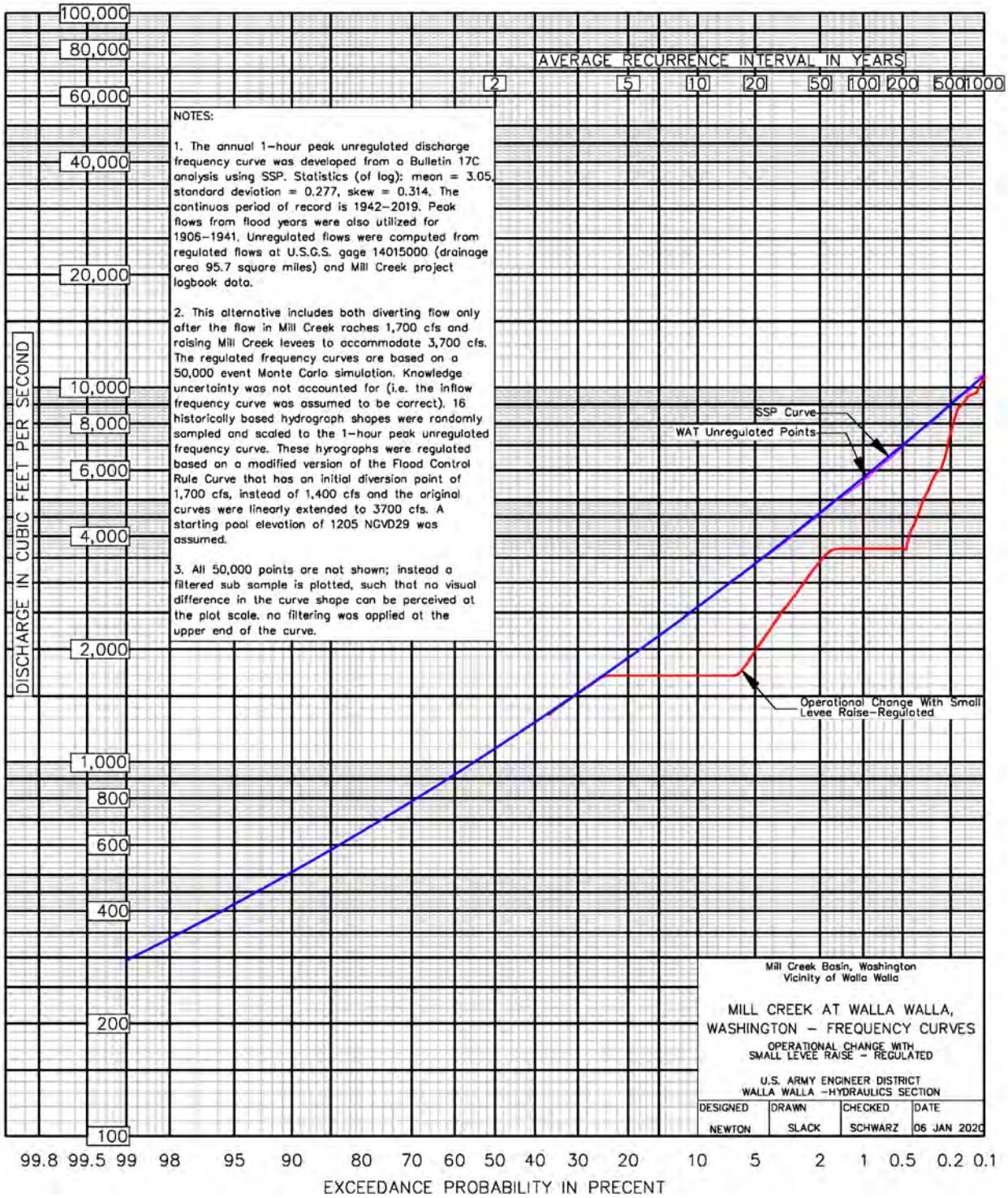
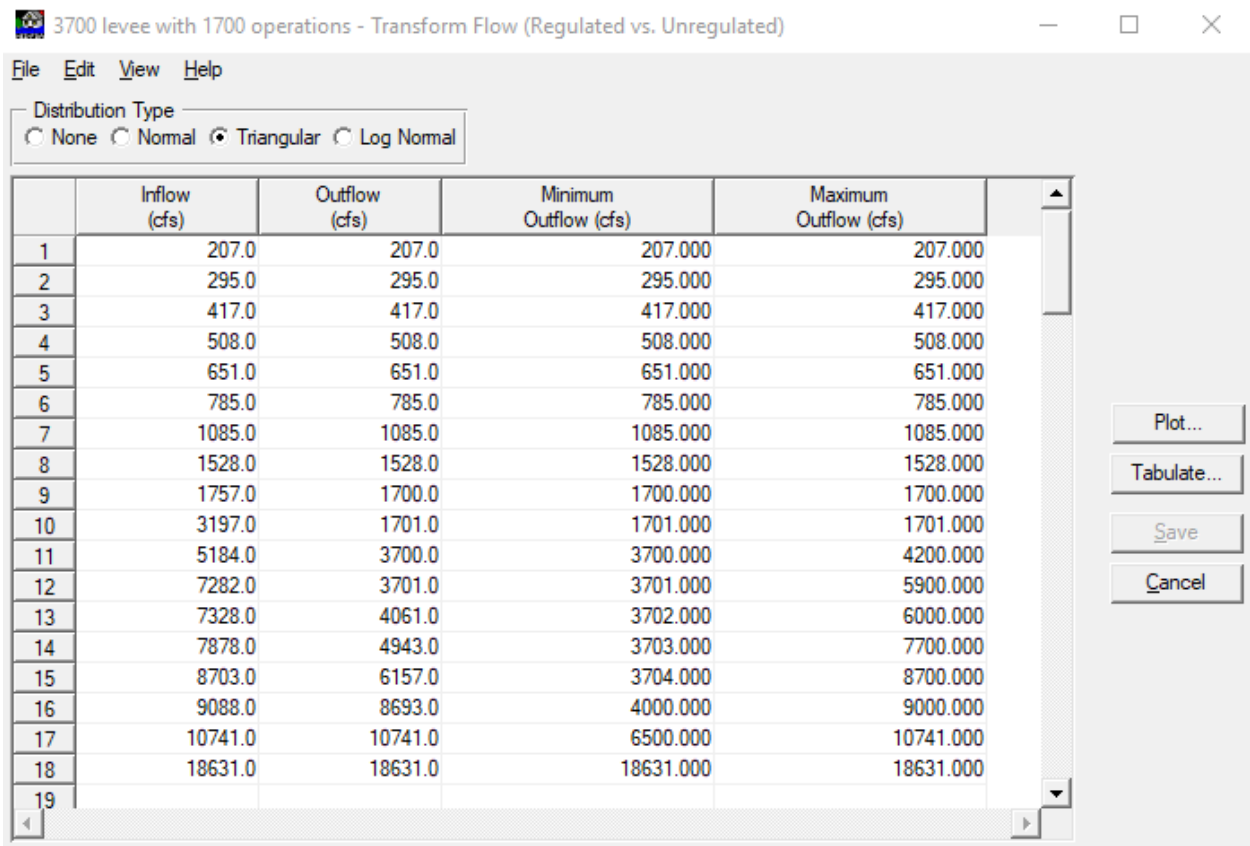


Figure 5-3. Small Levee Raise and Operations Change Flow Frequency Curve

This operation maintains flows to the channel flow target of 3,700 cfs up to the 0.458-percent chance flood (218 Year).

The unregulated frequency curve was entered into HEC-FDA as an Analytical Type with Log Pearson III Statistics: Mean of 3.05, Standard Deviation of 0.2777 and skew of 0.314 with an equivalent record length of 78 years.

The conversion of unregulated to regulation was entered as Transform with a Triangular Distribution Type, see following Figure. This was based on a scatter point plot from HEC-WAT output.



	Inflow (cfs)	Outflow (cfs)	Minimum Outflow (cfs)	Maximum Outflow (cfs)
1	207.0	207.0	207.000	207.000
2	295.0	295.0	295.000	295.000
3	417.0	417.0	417.000	417.000
4	508.0	508.0	508.000	508.000
5	651.0	651.0	651.000	651.000
6	785.0	785.0	785.000	785.000
7	1085.0	1085.0	1085.000	1085.000
8	1528.0	1528.0	1528.000	1528.000
9	1757.0	1700.0	1700.000	1700.000
10	3197.0	1701.0	1701.000	1701.000
11	5184.0	3700.0	3700.000	4200.000
12	7282.0	3701.0	3701.000	5900.000
13	7328.0	4061.0	3702.000	6000.000
14	7878.0	4943.0	3703.000	7700.000
15	8703.0	6157.0	3704.000	8700.000
16	9088.0	8693.0	4000.000	9000.000
17	10741.0	10741.0	6500.000	10741.000
18	18631.0	18631.0	18631.000	18631.000
19				

Figure 5-4. Small Levee Raise and Operations Change Unregulated to Regulated Transform

This approach differs from the approach in SECTION 3 - Full Modeled Alternatives. In an effort to better define uncertainty, entering the unregulated frequency with uncertainty and the conversion to regulated with uncertainty, results in a more accurate combined uncertainty.

5.3 INUNDATION MODELING

5.3.1 HEC-RAS Model Development

The HEC-RAS model for the Interim Analysis used the same model extents, terrain data, LiDAR data, 2D grid area, and Manning's n values as those mentioned earlier in this report.

There were many improvements to the Interim model. The model was converted from HEC-RAS version 5.0.6 to version 5.0.7. The most notable improvement to the model was having

better bridge and tunnel measurements. Hand measurements were taken at these structures allowing for much more certainty in the model results. All the 2D grid cells facing lateral structures were changed to point 90 degrees from the lateral structures. This allowed the 1D to 2D transition to run faster and more stable. The lateral structures representing levees were reduced to be under 500 points. With less points the model computed the water going over the levees faster. Having less than 500 points allows the National Levee Database to be updated more easily sometime in the future. Many of the 1D cross sections from the mouth of Mill Creek to about 5.5 miles upstream had unnecessary ineffective flows causing the model to go unstable. These ineffective flows were taken out and replaced with a few permanent ineffective flows which made the model more stable. From the mouth of Mill Creek to 5.5 miles upstream was estimated to have a water depth of 1.5 feet below what the LiDAR showed. No river survey was available.

Bridge ineffective flows were added to account for the abutments and bridge ends. This made a big improvement in model stability and accuracy as water flowed around the structures. A couple missing foot bridges were add into the model. The tunnel (downtown area) was changed to be modeled as a lid instead of a culvert. This made the resulting flows more reasonable without being constrained to the culvert equations. Two openings were added to the tunnel section. One opening was just north of Main St. and another opening was from a parking lot failure just south of Rose St.

Along with existing conditions, Alternative 3700 cfs levee raise (small) with 1700 cfs initial diversion operations change was also modeled. The left side levee (facing downstream) from the railroad crossing south of 9th Ave to 9th Ave was raised 0.7 feet. The left side levee from Tausick Way to Division Works (Yellowhawk creek) was raised 1.0 feet.

Modeled Scenarios for (1) Existing Conditions and (2) 3700 Small Levee Raise with 1700cfs Initial Diversion Operational Change:

Non-failure

Building (tunnel) collapse with 90% blockage

Parking lot collapse near Rose Street with 90% blockage

Wall failure near Otis with 50% blockage

Levee failure (from overtopping) upstream of Tausick Way

Levee failure (from overtopping) upstream of Division Works (Yellowhawk Creek)

Modeled Flows:

10604cfs, 9371cfs, 7171cfs, 5005cfs, 3700cfs, 3500cfs, 1700cfs, 1400cfs

5.3.2 Levee Sizing for Levee Raise and Certification

It was determined in SECTION 6 - Stabilizer Failure assessment that the stabilizers would be able to accommodate higher flows than 3,500 cfs without expected failure. Based on that levee raises were considered as an alternative for this study. This section discusses the approach, analysis, and assumptions that went into sizing levees for the levee raise with operations change alternative that meets levee certification standards. This analysis follows hydrology and hydraulic requirements outlined in ECB 2019-11 (EC 1165-2-218 Draft). This alternative considered a future levee raise targeting a 3,700 cfs 1-percent exceedance flood coupled with a change in the initial diversion to 1,700 cfs. The primary tool used for this analysis was HEC-FDA.

5.3.2.1 Alternative Small Levee Raise and Operations Change

To evaluate the existing conditions to meet hydrology and hydraulic characteristics required for levee certification by ECB 2019-11 (EC 1165-2-218 Draft) the following analysis and assumptions were made.

For this alternative the regulated flow frequency curve was revised to change the regulation target from 3,500 cfs to 3,700 cfs. In addition the initial diversion point was raised from 1,400 cfs to 1,700 cfs. The regulated frequency curve under this alternative shows the 1-percent chance flood as 3,700 cfs. The rest of this section outlines levee sizing to meet levee certification requirements for a 3,700 cfs levee.

<u>Flow Frequencies</u>

The unregulated frequency curve was the same as used in SECTION 3 - Full Modeled Alternatives

The unregulated and regulated flow frequency curve from HEC-WAT output is shown in Figure 5-3.

This operation maintains flows to the channel flow target of 3,700 cfs up to the 0.458-percent chance flood (218 Year).

The unregulated frequency curve was entered into HEC-FDA as an Analytical Type with Log Pearson III Statistics: Mean of 3.05, Standard Deviation of 0.2777 and skew of 0.314 with an equivalent record length of 78 years.

The conversion of unregulated to regulation was entered as Transform with a Triangular Distribution Type, see following Figure. This was based on a scatter point plot from HEC-WAT output.

3700 levee with 1700 operations - Transform Flow (Regulated vs. Unregulated)

File Edit View Help

Distribution Type
None Normal Triangular Log Normal

	Inflow (cfs)	Outflow (cfs)	Minimum Outflow (cfs)	Maximum Outflow (cfs)
1	207.0	207.0	207.000	207.000
2	295.0	295.0	295.000	295.000
3	417.0	417.0	417.000	417.000
4	508.0	508.0	508.000	508.000
5	651.0	651.0	651.000	651.000
6	785.0	785.0	785.000	785.000
7	1085.0	1085.0	1085.000	1085.000
8	1528.0	1528.0	1528.000	1528.000
9	1757.0	1700.0	1700.000	1700.000
10	3197.0	1701.0	1701.000	1701.000
11	5184.0	3700.0	3700.000	4200.000
12	7282.0	3701.0	3701.000	5900.000
13	7328.0	4061.0	3702.000	6000.000
14	7878.0	4943.0	3703.000	7700.000
15	8703.0	6157.0	3704.000	8700.000
16	9088.0	8693.0	4000.000	9000.000
17	10741.0	10741.0	6500.000	10741.000
18	18631.0	18631.0	18631.000	18631.000
19				

Plot... Tabulate... Save Cancel

Figure 5-5. Small Levee Raise and Operations Change Unregulated to Regulated Transform

This approach differs from the approach in SECTION 3 - Full Modeled Alternatives. In an effort to better define uncertainty, entering the unregulated frequency with uncertainty and the conversion to regulated with uncertainty, results in a more accurate combined uncertainty.

Hydraulic Model

For levee sizing the unsteady hydraulic model developed for inundation mapping of this interim analysis was adapted to be a steady flow model. The 2D overbank areas were removed and the model was utilized as a steady flow 1D channel only model for levee sizing and certification. Eight steady flow profiles were developed for the for specific percent exceedance flows as shown the following table. These were developed at key points in the regulated frequency curve.

Table 5-2. Small Levee Raise and Operations Change Frequency Flows, Levee Sizing

Probability	Flow (cfs)
0.5	1,085
0.1 (0.06588 – 0.2377)	1,700
0.02	3,500
0.01 (0.00458 – 0.01486)	3,700
0.00448	3,900
0.004	4,439
0.003	5,400
0.002	7,629

Channel n values were set to 0.04 for the levee reach.

Evaluation Reaches

In the leveed areas, evaluation reaches were identified between selected bridges. The number of independent reaches was increased with this Interim Analysis. Within the reaches index locations were selected based on the least available freeboard between the 1 percent chance flood profile (3,700 cfs) and the levee top for the lower of the Left or Right Levee. One or Two index locations were selected for each reach based on least available freeboard as an indicator of the critical index cross section. Where there were areas of low freeboard on both left and right levees in a given reach, two index cross sections were identified. The following table indicates the reaches, model reach names in HEC-FDA and index cross sections from the RAS model.

Table 5-3. Evaluation Reaches for Levee Sizing

Reach	Side of Critical Freeboard	Reach Name in FDA	Index Cross Section in RAS and FDA
Division Works to Dam	Left	1. Division to Dam	11.4919
Tausick to Division Works	Left	2. Tausick to Division Works	11.0901
Wilber to Tausick	Right	3. Wilber to Tausick	10.3589
	Left	4. Wilber to Tausick	9.9165
Rosevelt to Wilber	Right	5. Roosevelt to Wilber	9.8216
9th to Rail Road	Left	6. 9th to RR	7.3076
13th to Rail Road	Both	7. RR to 13th	7.1723
Myra to 13th	Right	8. 13th to Myra	6.4820
Gose to Myra	Right	9. Gose to Myra	6.2830
	Both	91. Gose to Myra	5.6062

Stage Uncertainty

In the HEC-FDA model, the Standard Deviation of the Error was determined by computation of uncertainty in friction and wave height. The manning's n values were varied in the HEC-RAS model to determine the standard deviation for the index locations for each flow frequency. The low channel manning's n was set to 0.03 and the high manning's n was set to 0.05, with the best estimate being 0.04. The following equation was used to determine the Standard deviation of error for each flows surrounding the design target at index cross sections.

Standard Deviation of the Error = [(high stage – expected stage)^2 + (low stage – expected stage)^2 /2]$^{1/2}$

Next, the equation results were compared to Table 5-2 in EM 1110-2-1619 to assure that these values met minimum standard deviation of error for stage as recommended in the EM. For these reaches the table indicated something between 0.7 and 1.3 would be a minimum value. Finally, to get smoothed results a single value was used for all flood frequencies at each index location based on evaluation of the calculated Standard Deviation of the Error and the EM guidance. This value was selected as the highest from flows between 3,500 cfs and 5,400 cfs as this is the critical flow range for levee sizing and certification or 0.7 from the EM. The reason for selecting a single value was based on lower confidence in the values obtained from the rough hydraulic model where instabilities have not all been worked out, and meeting the minimum range given in the EM. The following table includes the calculated values from the Standard Deviation of the Error Equation for the critical flow range used for levee sizing and levee certification calculations and the selected value.

Table 5-4. Standard Deviation of the Error for Index Cross Sections

Index Cross Section	Minimum Standard Deviation of the Error from EM 1110-2-1619	Calculated Standard Deviation of the Error	Selected Standard Deviation of the Error for Index Cross Section
11.4919	0.7	1.3	1.3
11.0901	0.7	0.9	0.9
10.3589	0.7	0.4	0.7
9.9165	0.7	1.1	1.1
9.8216	0.7	0.5	0.7
7.3076	0.7	0.9	0.9
7.1723	0.7	0.1	0.7
6.4820	0.7	0.6	0.7
6.2830	0.7	0.5	0.7
5.6062	0.7	0.9	0.9

Results

To meet levee certification requirements for hydrology and hydraulics outlined in ECB 2019-11 (EC 1165-2-218 Draft), a levee needs a Conditional Non-Exceedance Probability of greater than 0.85 for the 1% event at all index locations.

In order to size the levee raise to meet levee certification standards for the 1-percent chance flood (3,700 cfs), the levee from Myra to 9th Street Bridges will need to be raised 0.7 feet. The following table summarizes the results.

Table 5-5. Levee Raise Requirements

Critical Index Cross Section	Damage Reach	Conditional Non-Exceedance Probability 1% Exceedance Flood (3,700 cfs) Before Levee Raise	Levee Raise Required to meet 0.85 Probability for Levee Certification / Levee Side (feet)
11.4919	1. Division to Dam	0.8676	0.0 / Left
11.0901	2. Tausick to Division Works	0.7729	1.0 / Left
10.3589	3. Wilber to Tausick	0.96	0.0 / Right
9.9165	4. Wilber to Tausick	0.8594	0.0 / Left
9.8216	5. Roosevelt to Wilber	0.9997	0.0 / Right
7.3076	6. 9th to RR	0.7853	0.7 / Left
7.1723	7. RR to 13th	0.876	0.0 / Both
6.4820	8. 13th to Myra	0.9459	0.0 / Right
6.2830	9. Gose to Myra	0.9724	0.0 / Right
5.6062	91 Gose to Myra	0.9477	0.0 / Both

As the hydraulic model and hydrology are refined a more definitive determination can be made.

5.3.3 Inundation Mapping and FDA Input

Existing Conditions

The Existing Conditions scenario models Mill Creek as it appears today. No levee raises or excavations are modeled under this scenario.

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Table 5-6. Flows for Existing Conditions Scenario

Existing Conditions								
Return Period (yr):	1000	694	543	248	183	56 to 176	14	3
Probability:	0.001	0.00144	0.00184	0.004	0.0055	0.01758 to 0.00568	0.0728	0.34
Peak Flow (cfs):	10604	9371	8640	5005	3700	3500	1700	1400

Each of these flows were modeled and 2D overbank inundation maps were generated. This was completed for the following 5 scenarios:

1. No Failure
2. Levee Breach upstream Yellowhawk Division Structure
3. Levee Breach upstream Tausick Way Bridge
4. Channel Blockage upstream Otis Street Bridge
5. Channel Blockage in Tunnel exiting waters upstream Colville Street.

<u>Small Levee Raise and Operations Change Alternative</u>

Under the alternative levees would be raised to meet 1-percent annual chance exeedance flows (3,700 cfs) and the reservoir operations plan would increase the initial diversion to 1,700 cfs and the target release to 3,700 cfs.

Table 5-7. Flows for Small Levee Raise with Operations Change Alternative

Small Excavation								
Return Period (yr):	1000	694	543	295	67 to 218	50	10	3
Probability:	0.001	0.00144	0.00184	0.0034	0.01486 to 0.00458	0.02	0.1	0.34
Peak Flow (cfs):	10604	9371	8640	5005	3700	3500	1700	1400

Each of these flows were modeled and 2D overbank inundation maps were generated. This was completed for the following 5 scenarios:

1. No Failure
2. Levee Breach upstream Yellowhawk Division Structure
3. Levee Breach upstream Tausik Way Bridge
4. Channel Blockage upstream Otis Street Bridge
5. Channel Blockage in Tunnel exiting waters upstream Colville Street.

FDA Input

The output from the hydraulic analysis was provided for HEC-FDA use to determine economics. Locations for Functions coincide with Fragility Curve locations. These were determined as locations in the design channel most likely to fail based on a PFMA analysis described in the Engineering Appendix.

The Standard Deviation of the Error was determined by computation of uncertainty in friction and wave height. The manning's n values were varied in the HEC-RAS model to determine the standard deviation for the index locations for each flow frequency. The low channel manning's n was set to 0.03 for the levee reach, and 0.01 for the concrete channel. The high manning's n was set to 0.05 for the levee reach and 0.02 for the concrete channel. The best estimate being 0.04 for the levee reach and 0.015 for the concrete channel. The following equation was used to determine the Standard deviation of error for each flow surrounding the design target at index cross sections.

Standard Deviation of the Error = [(high stage – expected stage)^2 + (low stage – expected stage)^2 /2]$^{1/2}$

Next, the equation results were compared to Table 5-2 in EM 1110-2-1619 to assure that these values met minimum standard deviation of error for stage as recommended in the EM. For these reaches the table indicated something between 0.7 and 1.3 would be a minimum value. Finally, to get smoothed results a single value was used for all flood frequencies at each index location based on evaluation of the calculated Standard Deviation of the Error and the EM guidance. This value was selected as the highest from flows between 3,500 cfs and 5,400 cfs as this is the critical flow range for levee sizing and certification or 0.7 from the EM. The reason for selecting a single value was based on lower confidence in the values obtained from the rough hydraulic model where instabilities have not all been worked out, and meeting the minimum range given in the EM. Model inputs are shown in the following table.

Table 5-8. Stage-Discharge Function with Uncertainty

FDA Menu: Stage-Discharge Function with Uncertainty					
Function	**RAS Location**	**Physical Description**	**Distribution Type**	**Stage Where Error Becomes Constant**	**Standard Deviation of Error for Entered Stage**
8.1913	8.1913	Upstream Colville St. Bridge (tunnel)	Normal	955.92	0.7
8.7038	8.7038	Upstream Otis St. Bridge	Normal	983.86	0.7
10.968	10.968	Upstream Tausick Bridge	Normal	1140.17	0.9
11.4919	11.4919	Upstream Division Works	Normal	1175.68	1.3
	2D Area	Overbank Regions	Normal		0.7

Levee Features are shown in the following table for the following alternatives:

1. Existing Conditions
2. Small Levee Raise with 1,700 cfs initial diversion Operations Change

Table 5-9. Levee Features Existing Conditions

FDA Menu: Levee Features	
Levee Name (cross section Location)	**Top of Levee Stage (feet NAVD)**
8.1913	963.65
8.7038	990.89
10.968	1144.97
11.4919	1180.40

Levee Features are shown in the following table for the Small Levee Raise with 1,700 cfs initial diversion Operations Change.

Table 5-10. Levee Features for Small Levee Raise with Operations Change

FDA Menu: Levee Features	
Levee Name (cross section Location)	**Top of Levee Stage (feet NAVD)**
8.1913	963.65
8.7038	990.89
10.968	1145.97
11.4919	1180.40

5.4 MODEL SUMMARY RESULTS

Information in this section is to summarize results and compare alternatives. The following figure present the mean Annual Exceedance Probability (AEP) and Long Term Exceedance Probability (LTEP) computed for each alternative. The LTEP is the likelihood of exceedance at least once in the specified period and is computed as $1-(1-AEP)^N$, where N = number of years.

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Table 5-11. Levee Risk Assessment for Existing Conditions

Reach	Conditional Non-Exceedance Probability of Levee Overtopping for 1% Flood	Mean (Expected) AEP for Levee Overtopping	LTEP (Probability of Exceedance over Indicated Time)		
			10 Years	30 Years	50 Years
1. Division to Dam	0.8577	0.0063	0.0609	0.1717	0.2694
2. Tausick to Division Works	0.7608	0.0075	0.0725	0.202	0.3135
3. Wilber to Tausick	0.9518	0.0029	0.029	0.0846	0.137
4. Wilber to Tausick	0.8416	0.0067	0.0648	0.182	0.2845
5. Roosevelt to Wilber	0.9996	0.0003	0.0025	0.0074	0.0124
6. 9th to RR	0.7749	0.0072	0.0694	0.194	0.3019
7. RR to 13th	0.8604	0.0058	0.0566	0.1605	0.2529
8. 13th to Myra	0.934	0.004	0.039	0.1125	0.1804
9. Gose to Myra	0.9642	0.0029	0.0287	0.0836	0.1354
91 Gose to Myra	0.9405	0.0034	0.0333	0.0967	0.1559

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Table 5-12. Levee Risk Assessment for 3700 cfs Levee and Operations Change (After Raise)

Reach	Conditional Non-Exceedance Probability of Levee Overtopping for 1% Flood (after Raise)	Mean (Expected) AEP for Levee Overtopping	LTEP (Probability of Exceedance over Indicated Time)		
			10 Years	30 Years	50 Years
1. Division to Dam	0.8676	0.0058	0.0566	0.1603	0.2526
2. Tausick to Division Works	0.8821	0.0053	0.0519	0.1478	0.234
3. Wilber to Tausick	0.96	0.0024	0.0234	0.0685	0.1116
4. Wilber to Tausick	0.8594	0.0063	0.0614	0.1732	0.2716
5. Roosevelt to Wilber	0.9997	0.0002	0.0023	0.0067	0.0112
6. 9th to RR	0.8843	0.005	0.0486	0.1388	0.2205
7. RR to 13th	0.876	0.0054	0.053	0.1508	0.2385
8. 13th to Myra	0.9459	0.0035	0.0345	0.1001	0.1611
9. Gose to Myra	0.9724	0.0024	0.0236	0.0691	0.1124
91 Gose to Myra	0.9477	0.003	0.0297	0.0866	0.1401

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SECTION 6 - STABILIZER FAILURE ASSESSMENT

An extreme event with a flow rate higher than the 1996 event (3,800 cfs) creates high velocities in the middle of the channel. The streambed materials are erodible and begin to scour. A scour hole forms at the downstream toe of a stabilizer between station 321+00 and station 275+00.

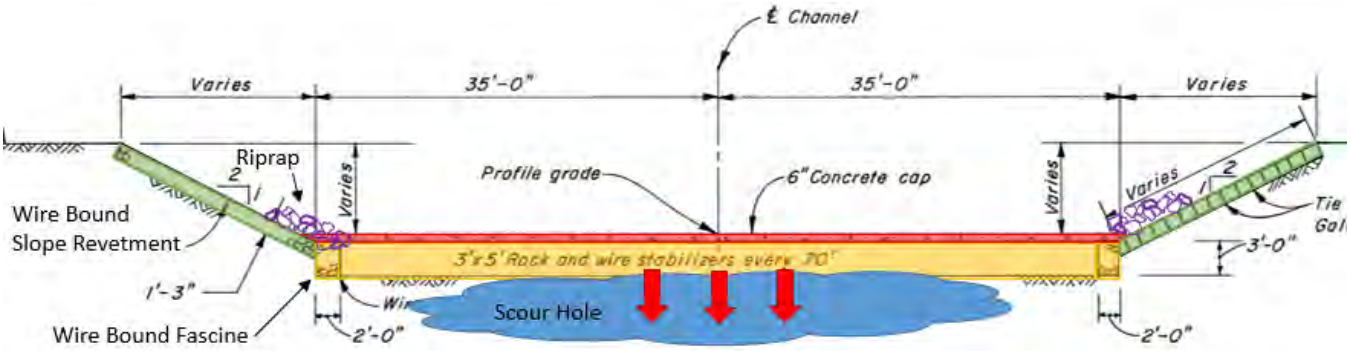

Figure 6-1. Cross-Section Showing Failure Mode

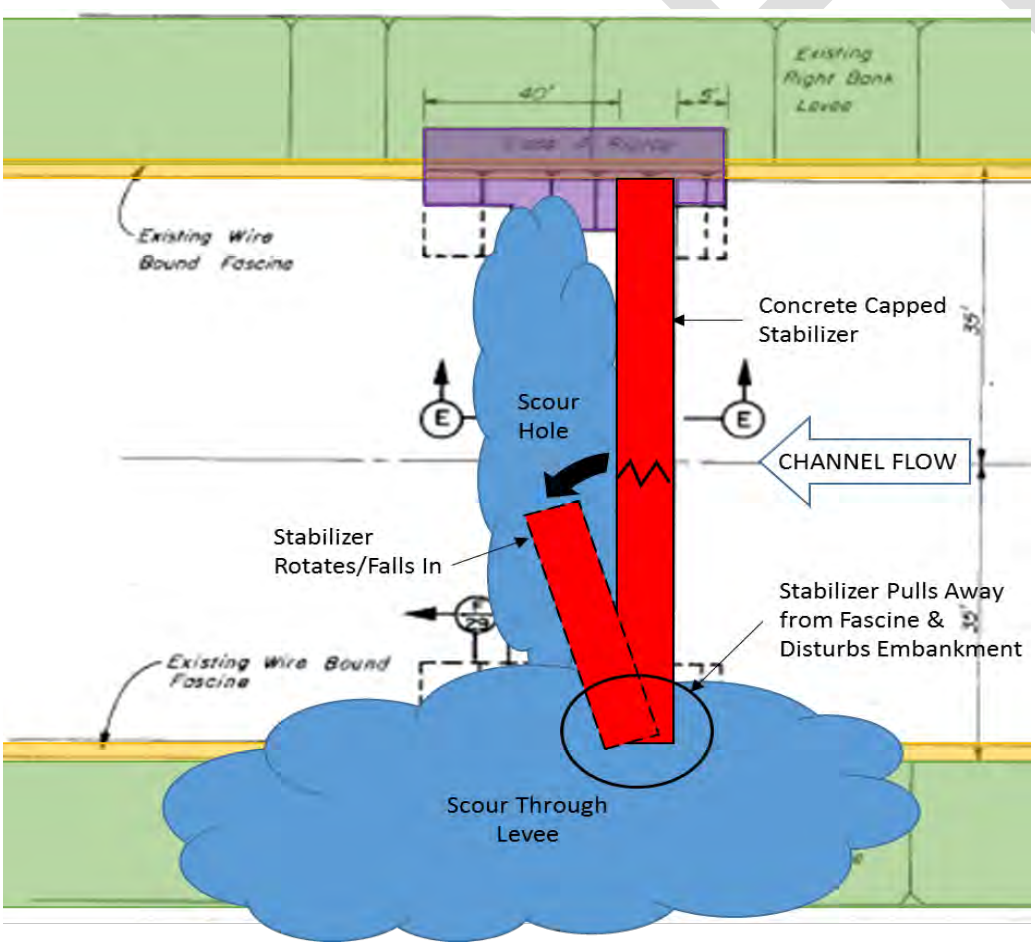

Figure 6-2. Plan View Showing Failure Mode

The scour hole widens and deepens to a point where the stabilizer is unsupported and the stabilizer fails into the scour hole. The scour hole and stabilizer failure expand laterally to the point where the stabilizer near the bank pulls away from the fascine and the movement causes disturbance and removal of the armoring along the water side of the levee. The exposed levee begins to scour and erode. The velocities and duration of the event are sufficient to continue erosion of the levee embankment. Intervention is unsuccessful. Erosion continues and progresses through the levee resulting in out of bank flow through the levee.

6.1 BACKGROUND

Most of the Mill Creek Flood Control Channel is either channelized or has a very low levee embankment height (typically between 0 and 3 feet). Assessment focused on the left embankment between stations 275+00 and 321+00 where the levee has the largest embankment height, about 9 feet. The left side (looking downstream) was chosen because the inundation extents were greater on the left bank for a failure of the levee system than the right bank. Even though this failure mode focuses on a small portion of the flood control system, stabilizers were constructed along the entire length of the system (except for the concrete channel section) and failure of the stabilizers could happen anywhere in the system, albeit with less potential consequences.

MC-239 Looking South - Construction Stabilizer in Improved Channel of Mill Creek.
10-23-40

Figure 6-3. Photo Showing Construction of Stabilizers

The channel was designed with stabilizers that span the width of the channel and are spaced 70 feet apart in the described area. Figure 6-3 shows a photo of the construction of a stabilizer. A

cross-section showing the original channel is shown in Figure 6-4, and a section of the original stabilizer is shown in Figure 6-5. The stabilizers were constructed with wire mesh and filled with gravel, cobbles, and/or rock fill and are approximately 3 feet wide by 5 feet high. The upstream face of the stabilizers are completely embedded in the river channel, while the downstream face is embedded approximately 1 foot. A wire bound fascine, 5 feet wide by 3 feet high, was constructed longitudinally along the entire toe of the levee embankment. The slope of the levee is protected with a wire bound revetment that is 15 inches thick. The material inside the slope revetment is mostly cobbles that range in size from 4 to 8 inches with some cobbles up to 10 inches, visually estimated from a site visit on March 21, 2011 (Figure 6-6). According to available construction drawings in Design Memorandum 7 (DM7), there is apparently no tie-in between the stabilizers and the fascine or the fascine and the slope revetment or the slope revetment and the levee embankment.

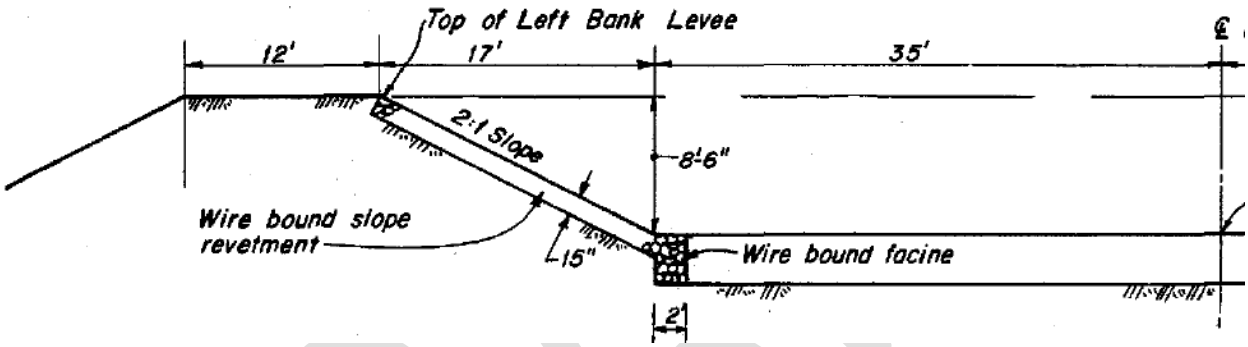

Figure 6-4. Original Channel Cross-Section
(DM7, Plate 3)

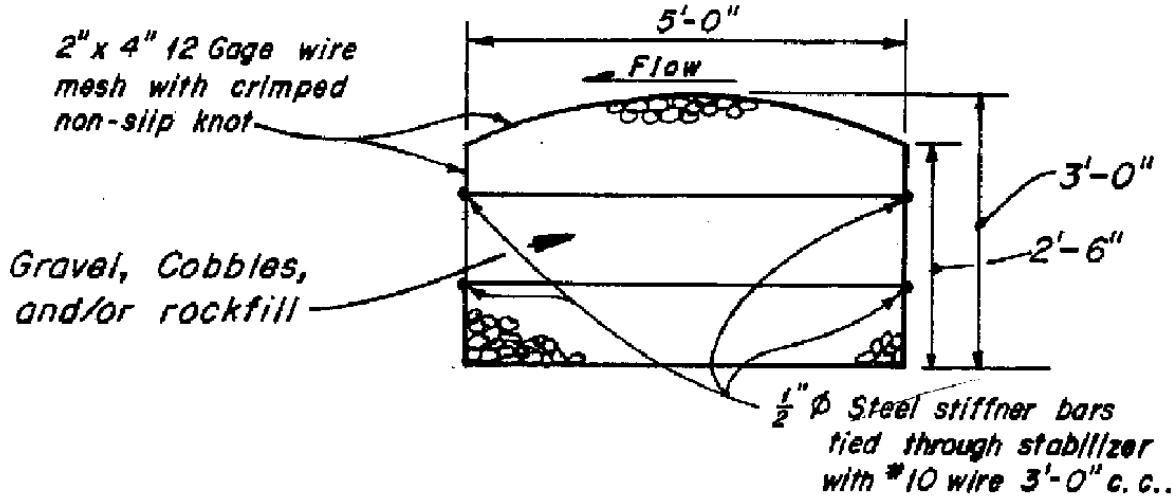

Figure 6-5. Original Stabilizer Cross-Section
(DM7, Plate 4)

Figure 6-6. Photo Showing Cobble Size in Slope Revetment (3/21/2017 Site Visit)

Gradation tests of borrow materials for the levee embankments were not performed during construction. There are loess deposits near the project, but loess is not thought to be used in the embankment, although there was no information available for the borrow source for the levee embankment. Construction photos suggest there is a significant percentage of gravel and cobbles with some fines present in the embankment materials. The channel bottom material consists of alluvial fan deposits which generally consists of cobbles, well-graded gravels with sand (GW) and poorly graded gravel with silt and sand (GP-GM) across the entire site.

The channel was originally designed for flows up to 5,400 cfs. In 1986, hydraulic modeling was performed by the Waterways Experiment Station (WES) to determine the maximum flow that can pass through the channel without failing the stabilizers. Numerical and physical model studies were performed to determine scour potential and provide design guidance for the stabilizers. A physical model was built to simulate the concrete capped stabilizers in the portion of the channel that is 70 feet wide with stabilizers spaced 70 feet apart. Samples of the prototype (Mill Creek Project) river bed materials were taken and scaled appropriately for the physical model. The gradations for the river bed materials are shown in Figure 6-7. When there is flow over the stabilizers, finer particles (sands and silts) are washed away leaving behind a thin layer of coarser material, referred to the armor layer in the investigation report. Using the Hjulstrom Curve (Figure 6-9), the velocities needed to erode sands and gravels are about 3 ft/sec and 8 ft/sec, respectively, which is exceeded during high flow events. The gradation curves for the armor layer is shown in Figure 6-8. Class A riprap was used in the model at the toe of the levee to prevent excess scour. The specified gradation for the Class A riprap is shown in Table 6-1. The riprap diameter in this table was estimated using equation 3-1 in EM 1110-2-1601. After the hydraulic model report was completed, Class A riprap was added

to the downstream toe of the stabilizers and the toe of the levee as shown in Figure 6-11 and Figure 6-13. Based on visual observation during the site visit, the average size of the riprap was estimated to be about 20 inches in diameter.

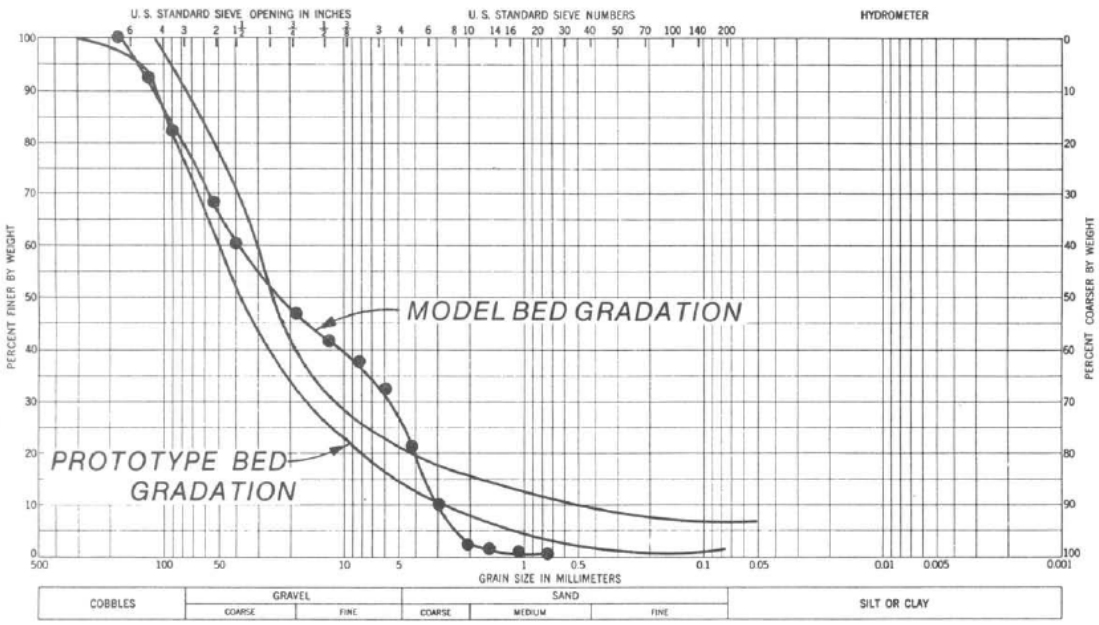

Figure 6-7. Model and Prototype Bed Gradations
(1986 Hydraulic Model Investigation, Plate 1)

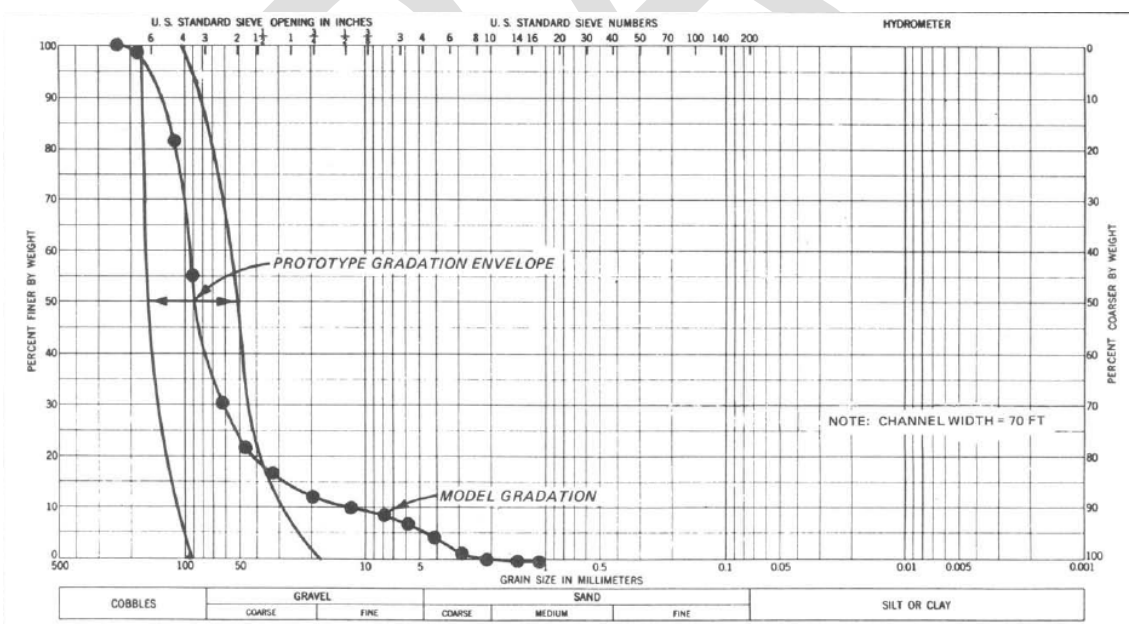

Figure 6-8. Armor Layer Gradation Curves
(1986 Hydraulic Model Investigation, Figure 6)

Table 6-1. Class A Riprap Gradations

Weight (lb)	Estimated Diameter (inches)	Percentage (%)
1,044 – 417	26.8 – 19.7	25
309 – 209	17.8 – 15.7	50
155 – 65	14.2 – 10.6	25

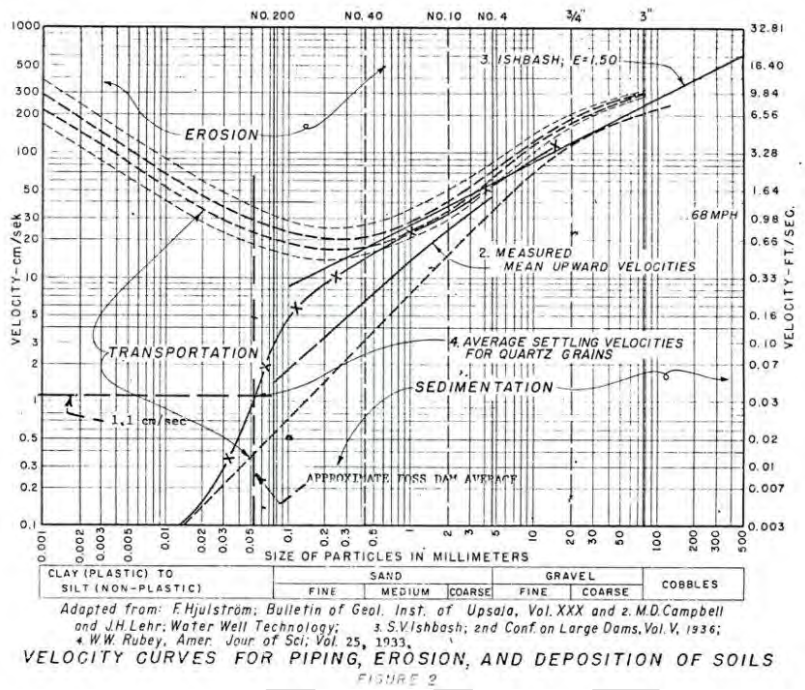

Figure 6-9. Hjulstrom Curve

The recommended design section for the stabilizers is shown in Figure 6-9. The modeling predicted that a 3,500 cfs flow would result in a maximum scour depth of 4.5 feet and an average scour depth of 4.3 feet. The duration of the peak discharge in the model was originally 24 hours but was shortened to 19 hours once it was observed that no discernible change occurred in the scour pattern. As stated in the 1986 Hydraulic Model Investigation, "A design discharge of 3,500 cfs was chosen because this discharge produced scour depths that fell into the range of allowable scour depths of 4 to 5.5 feet provided by the Walla Walla District."

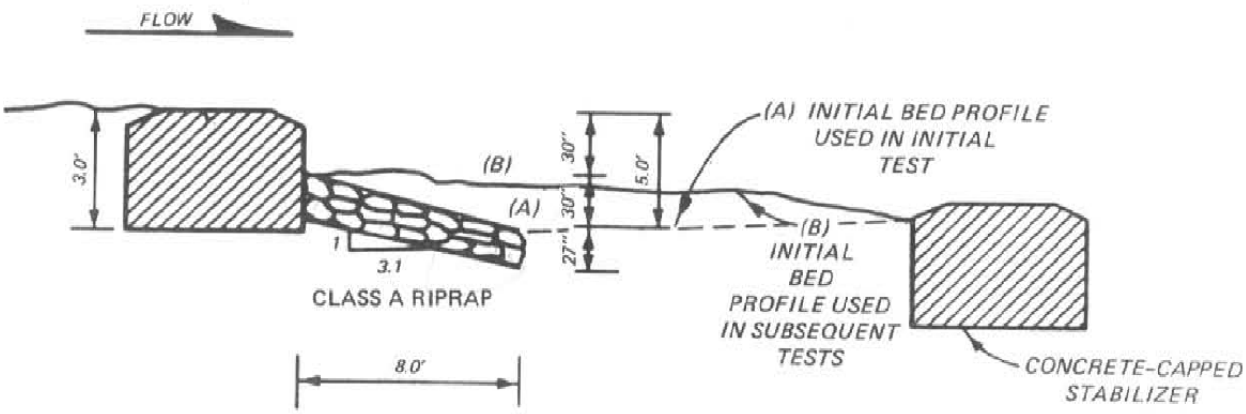

Figure 6-10. Recommended Design Stabilizer Protection
(1986 Hydraulic Model Investigation, Figure 8)

The stabilizers were repaired and capped with concrete over several contracts between 1951 and the late 1980's, after the 1986 Hydraulic Investigation. A section of the repaired stabilizers is shown in Figure 6-11. The concrete cap was placed on top of the stabilizers and a short cutoff was constructed on the downstream face of the stabilizers. The concrete cap ties into both the wire bound fascine and the wire bound slope revetment. Design drawings in DM7 show that cap was cast in place, and only the downstream face of the cap is reinforced (Figure 6-12). The top and upstream face of the stabilizer are not reinforced. Class A riprap was placed at the downstream end of the stabilizer and is 8 feet long and 27 inches thick (Figure 6-13). This helps to prevent scour at the downstream end of the stabilizer. Class A riprap, which is typically greater than 20 inches wide, was also placed at the toe of the levee embankment, covering the fascine and rising two feet up the slope of the embankment.

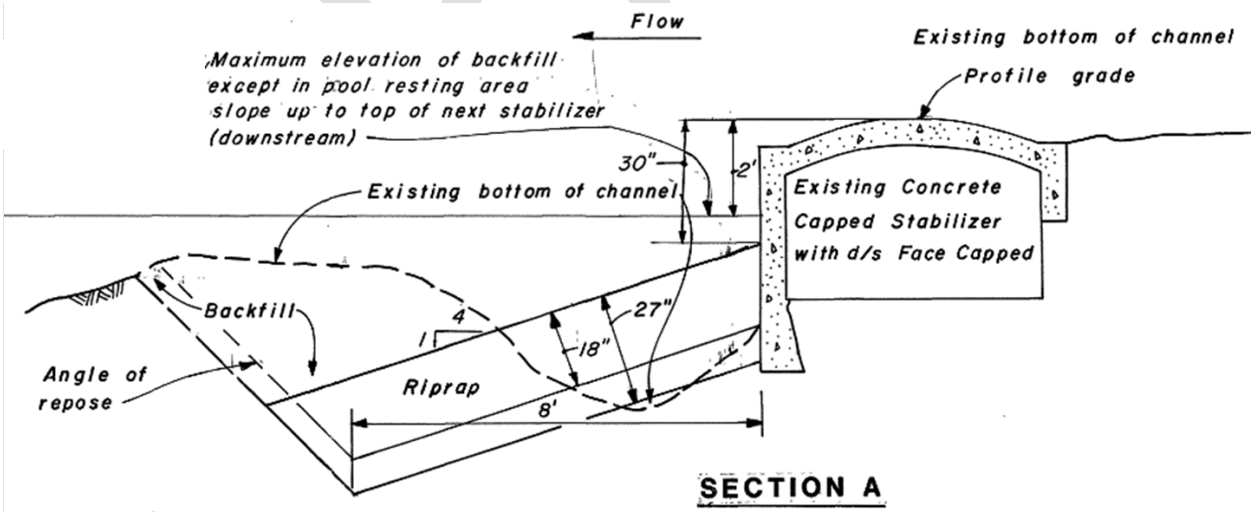

Figure 6-11. Cross-Section of Concrete Capped Stabilizers
(Mill Creek Rehabilitation Phase III, Sheet 10)

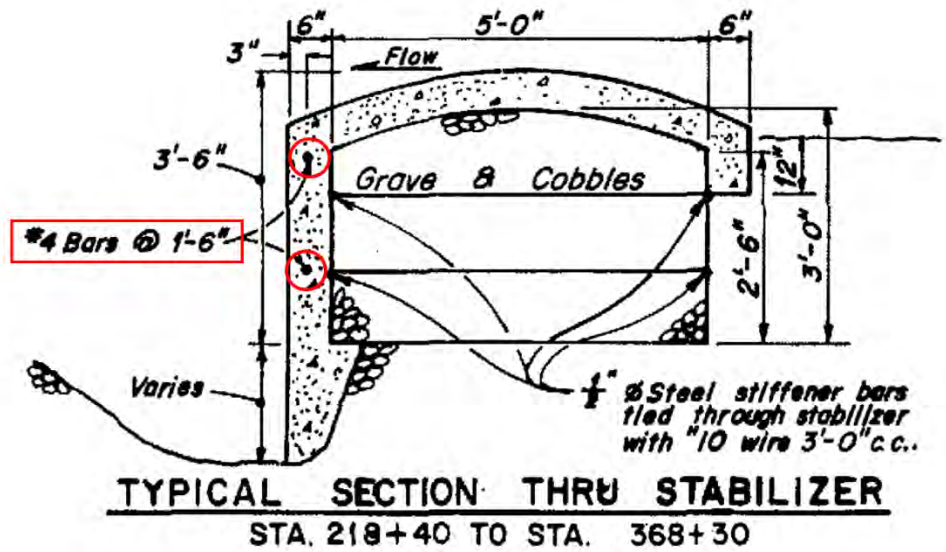

Figure 6-12. Section through Stabilizer Showing Reinforcement on Downstream Side
(DM 7, Plate 10)

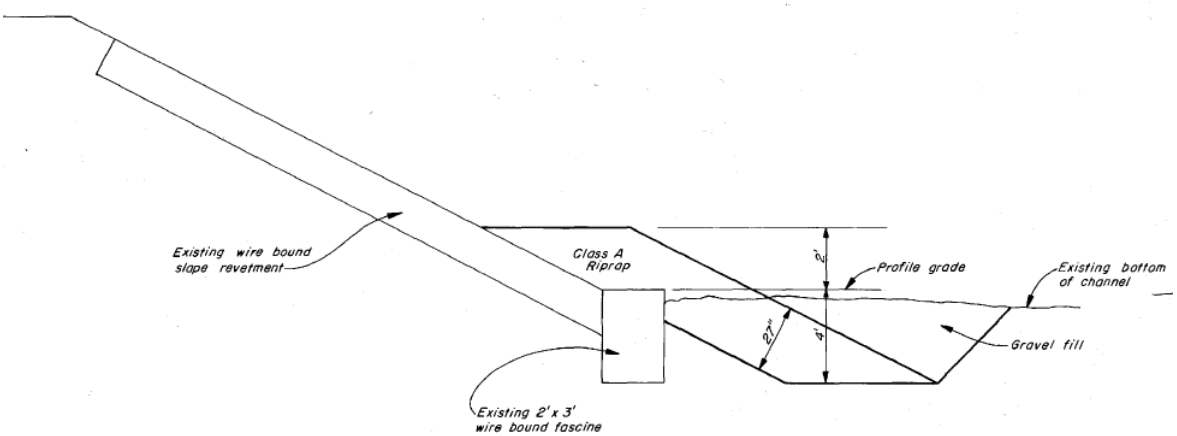

Figure 6-13. Cross Section Showing Riprap at Levee Toe
(DM7, Plate 29)

The 2017 RAS model shows a maximum outflow from the upstream diversion dam of 5,700 cfs and, in the location of assessment, a maximum velocity of approximat6ely 14 fps with an average velocity of approximately 10 to 12 fps in the center of the channel. The event duration is approximately 60 hours. Using the Plate B-37 (Figure 6-14), the D30 of the riprap should be designed to 12 inches for a velocity of 14 and depth of 9 feet. Based on this size, the Class A riprap at the toe of the stabilizers and toe of the levee is large enough to withstand the maximum velocity of 14 fps.

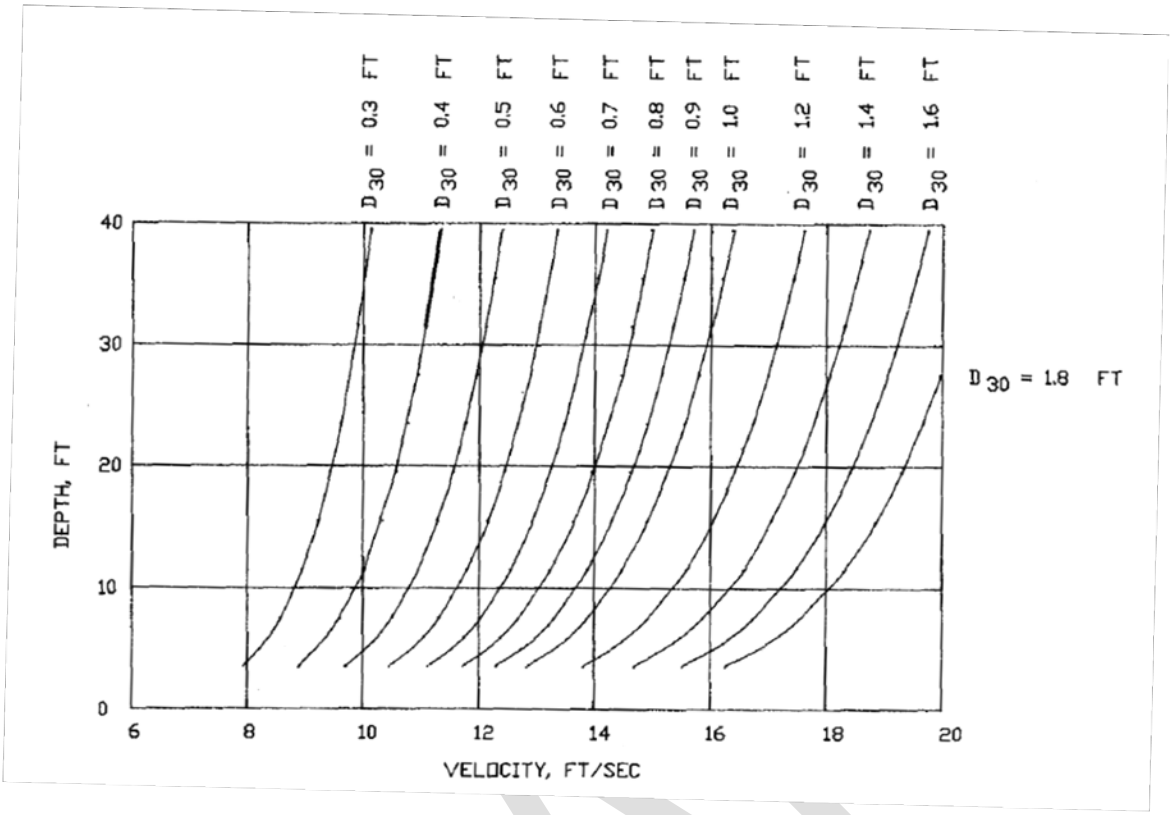

Figure 6-14. Depth-Average Velocity vs D30 and Depth
(EM 1110-2-1601, Plate B-37)

6.2 PERFORMANCE

The Mill Creek Levee system has over 300 stabilizers from upstream to downstream (station 370+12 to station 5+50). To date, all of the original wire-wrapped stabilizers have been either capped with concrete or replaced with sheet piles. Prior to capping the stabilizers with concrete, DM7 indicates that there were failures in locations and the material in the wire bound stabilizers was displaced downstream. To date, there has not been any indication of failure of the concrete capped stabilizers, although there has been minimal damage documented with a few stabilizers since they were capped. Damaged stabilizers have been repaired as needed by USACE and the flood control district. Figure 6-15 shows damage to one of the stabilizers due to the concrete cap being undermined. This stabilizer has since been repaired.

Scour at the downstream toe of the stabilizers was reported in the 1986 Hydraulic Investigation Report. A field sampling and testing program was performed as part of the 1986 Hydraulic Model Investigation. As part of this testing program, profile surveys were done to determine the actual depths of scour in the channel. Scour depths up to 6 feet were reported. The depths of scour reported were likely the cumulative result of multiple events over the project history.

An important factor in determining scour hole extents and depth is the trajectory of the main flow jet just downstream the drop edge. The jet either dives or rides along the surface. This

flow state may change with changes in discharge and tailwater depth. The flow state will not necessarily be the same for all of the Mill Creek stabilizers due to local channel geometry and inconsistent drop heights from one stabilizer to the next. Diving-jet flow can produce scour depths an order of magnitude deeper than surface-jet flow over similar durations, and diving-jet scour holes form closer to the structure (Newton, 2014). Newton observed that when flows are near the regime transition point scour initiates at about 5 drop heights downstream of the structure for surface flows and at about 2 drop heights for diving flows. He also observed that the maximum scour depth tends to occur near this initiation point. Surface flow scour backfills sediment toward the drop, while diving flow pushes sediment downstream. If the maximum scour hole depth is 5 drop heights downstream and sediment is being transported upstream, one might at first be tempted to completely discount surface-jet scour and conclude that it does not adversely affect the structure. However, at Mill Creek the stabilizers were designed such that the water surface curves downward at the stabilizer drop resulting in surface-jets being accompanied by a downstream surface wave. Newton (2014) observed that just downstream of the first wave trough the scoured bed responds to the wave constriction, creating a depression in the backfilled slope. This lessens protection that the backfilled slope might have otherwise provided.

At Mill Creek the stabilizers experience both diving and surface flow scour, but the flow state varies with discharge. At very high discharges the vast majority (if not all) stabilizers will be in the surface flow regime. This drastically reduces the risk of structures being undermined by scour at high flow, due to the backfilling nature of surface-jet scour. However, as the flow recedes at lower discharges it transitions back to a diving flow and the backfilled slope encourages the flow to dive at higher discharges (Newton, 2014).

If maximum discharge were to be increased by a small fraction one would not expect the structure to be noticeably more likely to be undermined by scour (due to the backfilling nature of the surface flow). However, no analysis has been done to determine if the additional hydraulic forces and flow vibrations would adversely impact the stabilizers structural integrity. Structural analysis of the stabilizers should be conducted before subjecting the stabilizers to increased discharges. It should also be noted that if any of the stabilizers were in a diving flow state, they would be subject to more risk from bed scour. An analysis should be conducted to determine which stabilizers, if any, might experience a diving flow at higher discharges.

Field observations indicate diving flow during 400 cfs on a small number stabilizers (the majority had a surface-wave flow). When the flow was near 1,400 cfs observations indicated a sole stabilizer upstream of the project office with diving flow (weir 95 from the 2009 survey). Downstream of the project office the stabilizers were not scrutinized as heavily. At 1,400 cfs the majority of stabilizers still experienced surface-wave flow, but flow was diving at a handful of stabilizers. A few of these stabilizers were located along the narrower 70 ft wide region downstream of Tausick Way, adjacent to the Mill Creek Sports Complex. The flow at those locations had a more typical hydraulic jump-like appearance. This likely implies that the channel/drop is not deep and that the channel is heavily armored, but this was not verified. If the 1,400 cfs inflection point on the Flood Control Rule Curve were moved to a higher discharge

the forces exerted on the armoring would increase and it would be prudent to reassess the armoring at all locations where the flow dives at 1,400 cfs.

The visible wire in the wire bound slope revetment appears to be in good condition as seen during the SQRA site visit. However, the toe of the revetment and the fascine are covered in riprap and submerged in some areas. In addition, the condition of the wire in the stabilizers prior to capping with concrete is unknown. It is possible that there is some corrosion of the wire that would allow loss of levee and slope materials as well as instability of the concrete cap of the stabilizers. Letters from the Walla Walla County Public Works (2009 and 2013 to 2017) indicate that damaged wire mesh on the channel embankments was repaired.

Figure 6-15. Historic Damage to Stabilizer
(Photo Taken 09/25/2012, Prior to Repair)

In 1996, a flood event produced a maximum discharge through the upstream diversion dam of 3,800 cfs (4,190 cfs instantaneous based on USGS gages) with maximum velocities estimated to be about 12 ft/sec. Based on Table 3-5 in the Mill Creek Diversion Dam SQRA Report, this event was determined to be close to the 1/100 year event for the project. The velocity was estimated with the RAS model taking the highest velocity predicted near the center of the channel within the area of this failure mode (station 275+00 and station 321+00). The Mill Creek Levee system had good performance overall with no loss or shifting of stabilizers during the 1996 event. Minor damage to the riprap was reported in some areas, which was subsequently repaired. In general, project and district personnel reported that the Federal section of the system performed better than the non-federal section. However, substantial erosion occurred at the upstream end of the system near the diversion dam due to flow over the spillway with some accumulation of soil material and debris reported on the levee in this area. There is a possibility that the material seen on the slope of the levee was actually debris and sediment

that was washed down from upstream of the diversion dam and was not the result of the levee slope eroding. This area has since been repaired and armored. After the 1996 event, there were limited inspections within the non-federal section and there was no After Action Review performed. This inspection was limited to the Mill Creek Flood Control District personnel walking along the levee. No inspection or survey was performed within the channel to determine depth of scour.

6.3 INTERVENTION

The Mill Creek Flood Control Channel is accessible (from multiple directions) during flood events, and the possibility of intervention is high. The area is visible and within a populated area, allowing detection to be more likely. If a stabilizer were to fail, there is not much that can be done to middle of the channel during the event, as flows will be high. However, if the embankment starts to erode and wash away, material could be end-dumped or pushed into place to slow down or stop the erosion. The top of the levee embankment can support vehicular traffic allowing construction vehicles access to the area if needed.

<u>Failure Factors</u>

Table 6-2. Summary of Likelihood Factors

More Likely Factors	Less Likely Factors
• There has been evidence of scour hole formation on the downstream side of stabilizers during past events • The size the scour holes downstream of stabilizers are unknown. Scour holes were not surveyed after the 1996 flood event. • Wire in gabion and fascine along levee could be corroded and absent in some places, allowing loss of levee and slope materials and creating instability of concrete cap • Potentially erodible soils in some areas beneath stabilizers that aren't armored • The submerged wire is likely to be in good condition, which could allow for the stabilizer to move and disturb the levee slope	• **Riprap is adequately sized based on the maximum velocity of 14 fps that was calculated in RAS.** • **Good performance history to date since concrete cap placement on stabilizers – survived the 100-year event in 1996 with minimal damage** • Stabilizers with greater than 18 inch drop were armored on the downstream side • **Stabilizer is not expected to move as a monolithic structure (would instead break up) and would therefore be less likely to damage fascine** • Maintenance history is good, records maintained by sponsor • Detection is likely and intervention is possible in most areas along the levee

	• Wire in stabilizer that is corroded and absent in places would allow stabilizer to move without disturbing the levee slope • There are likely cobbles and less erodible channel bed soils in some areas • ACE is 0.005 for 4,200 cfs (200-year event), 0.004 for 5,000 cfs (250-year event)

Note: Key factors are shown in **bold**

6.4 CONCLUSION

The hydraulic loading thought to be critical for this system is 5,000 cfs (ACE 0.004), which is higher than the project has seen in the past. A scour hole has to form and be large enough to fail a stabilizer, which was thought to be unlikely to occur in one event. The failure would have to widen and extend to the sides of the channel and pull the stabilizer away from the embankment. This failure progression was also thought to be unlikely as the fascine wire would likely fail locally, resulting in local unravelling of the stabilizer, rather than move as a rigid beam resulting in rotation and erosion at the connection with the fascine and embankment. Due to the lack of tensile reinforcement, monolithic behavior is not expected during failure of a stabilizer. The stabilizer is expected to break up relatively quickly and will not stay as a block, limiting the likelihood of rotation. Maximum predicted velocities (14 ft/sec) are not expected to exceed critical velocities (16 ft/sec) required to erode the Class A riprap at the base of the embankment and the duration of the event is relatively short (about 60 hours). Overall, performance of the stabilizers has been good throughout the project history. During the 1946 event, some of the original stabilizers broke up and failed locally but the levee was not significantly damaged. The stabilizers have since been capped by concrete, and performed well during the near 100-year event in 1996.

The 1986 Hydraulic Model Investigation shows that flows above 3,500 cfs would cause scour hole depths greater than acceptable (more than 4 to 5.5 feet deep). However, the 1996 event had a peak outflow of 3,800 cfs, and the system performed well, although scour depths from this event were not recorded. Structural analysis of the stabilizers should be conducted before subjecting the stabilizers to increased discharges. At higher discharges the vast majority (if not all) stabilizers will be in the surface flow regime. This drastically reduces the risk of structures being undermined by scour at high flow, due to the backfilling nature of surface-jet scour. An analysis should be conducted to determine which stabilizers, if any, might experience a diving flow at higher discharges.

The condition of the wire in both the stabilizers and in the fascine are unknown as they cannot be seen. There is limited information on the soil conditions and erodibility of the channel and embankment soils.

Mill Creek Prototype low flow modifications made to sills 53, 54 and 55 constructed in the federal reach of Mill Creek in July through September of 2012 and by others referred to as Tausick wiers in 2011 are not anticipated to increase scour or flow instability. These modifications allow for energy dissipation to occur over an extended foot print. These reinforced concrete structures will likely improve energy dissipation of the channel at higher flows.

Figure 6-16. Photo of 1996 Flood – Estimated at 3,800 cfs

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SECTION 7 - CLIMATE CHANGE SUMMARY

The following is a summary of the conclusions of the climate change qualitative analysis for the Mill Creek basin. For the full analysis see the Annex.

In the Mill Creek Basin, trends are noted for increasing water temperature, earlier hydrograph timing (decreased April 1 SWE), increased frequency of winter flows, decreased magnitude and longer duration summer flows. Various studies disagree about an increase in extreme precipitation with the majority leaning toward increased frequency (not magnitude) of extreme precipitation events, though variability is reported for precipitation generally. Specific magnitude and patterns of climate change impacts are not specified in this qualitative analysis. Through use of the online tools provided by USACE, no trend is apparent for the unregulated peak flows used as the basis for the Mill Creek GI. Based on the Middle Columbia HUC4 vulnerability analysis and literature review for the Northwest region, it can reasonably be projected that Mill Creek ecosystem will need to adapt or restructure in response to the gradual trend of warmer water temperatures, increased winter flow frequency, and less flow in the warm season. Additionally, the project may need to be able to handle a higher frequency of extreme precipitation events. Each business line considered high priority for Mill Creek are considered below individually.

- **Flood Risk Reduction**

  Flood control is the primary reason for the Mill Creek project. From the tools mentioned in this study Mill Creek appears most vulnerable to increased flood frequency. If the trend of increased frequency of flood events continues project structures may experience more frequent loading.

- **Ecosystem Restoration**

  Another business line relevant to the Mill Creek GI relates to water quality, stream fish habitat, and other ecosystem considerations. Project operations may need to adapt in consideration of salmonid habitat, flow magnitudes appropriate for fish passage, stream and shoreline habitat, and recreation opportunities. As far as Mill Creek GI scope allows, project alternatives may do well to incorporate elements that counteract the effect of increased temperature, lower flow or shifted hydrograph, and increased frequency of extreme events. Such measures could incorporate riparian vegetation for increased shading during typical flow magnitudes and perhaps even refuge in high flow events depending on how vegetation interacts with inundated hard structure. Further measures that could perhaps counteract an increasing temperature could include increased pool depth, or minimized shallow, slow flow exposed to direct sunlight. In light of these considerations, perhaps a more concentrated, faster and deeper channel would maintain lower channel temperature and improve valued species habitat.

- **Recreation**

  Lower base flows and increased flow variability may affect current patterns of recreation in and around the Mill Creek project. Timing of filling Bennington may need to be considered if the regional earlier runoff hydrograph timing pattern is expressed in the Mill Creek watershed.

Future with-project conditions will likely have no foreseeable significant effect on climate change at the scale considered in this analysis when compared to the future without-project conditions. No new large reservoirs or wetlands will be created as part of the proposed project, and the project is on a different scale than what would impact the regional climate. However, the apparent and projected trends showing increasing temperature, earlier hydrograph timing, decreasing base flow magnitude, and increasing frequency of high flow events will probably effect the project. Timing may shift to an earlier peak, runoff from precipitation events may occur more frequently in winter, and in-channel ecosystem may need to adjust to a gradually warmer average temperature. Although the Northwest is relatively invulnerable to these changes compared to other U.S. regions as indicated by the above vulnerability analysis, resiliency to these effects of climate change in the Mill Creek watershed can be incorporated into the chosen Mill Creek GI project alternative. Alternatives connected with resiliency would be a benefit to the project whether or not conditions at the site shift over time due to climate change.

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SECTION 8 - MODEL DESCRIPTIONS

The following are descriptions of the models used in this analysis.

MATLAB

MATLAB (matrix laboratory) is a programming language with a desktop numerical computing environment. It is commonly used by engineers and mathematicians because of its built in algorithms and efficient implementation of matrix and array mathematics. Code was developed using this tool to do flood frequency and reservoir routing, for rapid assessment of alternatives. This was utilized for work leading to the AMM report as well as the trade-off analysis in this report.

HEC-SSP

HEC-SSP (Hydrologic Engineering Center Statistical Software Package) was used in this study to develop the unregulated flood frequency curve for Mill Creek. This is U.S. Army Corps of Engineers developed software.

HEC-WAT

HEC-WAT (Hydrologic Engineering Center Watershed Analysis Tool) was used in this study to do random sampling of inflow frequencies, to scale hydrographs, to run the HEC-ResSim model, and develop regulated frequency curve data. This is U.S. Army Corps of Engineers developed software.

HEC-ResSim

HEC-ResSim (Hydrologic Engineering Center Reservoir Simulator) was used to model reservoir operations for existing conditions and the alternatives. This is U.S. Army Corps of Engineers developed software.

HEC-RAS

HEC-RAS (Hydrologic Engineering Center River Analysis System) was used for hydraulic modeling of the river channel and overbanks from Diversion Dam to the mouth of Mill Creek. The model was used for existing conditions and altered for the levee raise alternatives. This is U.S. Army Corps of Engineers developed software.

HEC-FDA

HEC-FDA (Hydrologic Engineering Center Flood Damage Analysis) was used to size levees for the levee raise alternative. It is also the model that was used for economic analysis along with H&H and Geotech data. This is U.S. Army Corps of Engineers developed software.

Additional information about these models can be found at: https://www.hec.usace.army.mil/.

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SECTION 9 - REFERENCES

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MILL CREEK FLOOD RISK MANAGEMENT GENERAL INVESTIGATION

Draft Feasibility Study and Environmental Assessment Report

APPENDIX A, HYDROLOGY AND HYDRAULICS
ANNEX, CLIMATE CHANGE SUMMARY

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DEPARTMENT OF THE ARMY
WALLA WALLA DISTRICT, CORPS OF ENGINEERS
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Climate Change Assessment

Mill Creek General Investigation Feasibility Study and Integrated Environmental Assessment

Hydraulics and Hydrology Branch

Northwest Walla Walla District, Northwestern Division

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COVER IMAGE:

Mill Creek watershed at the HUC10 level.

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Qualitative Assessment of Climate Change Impacts: Mill Creek General Investigation

1. Background

United States Army Corps of Engineers (USACE) projects and operations have generally proven robust enough to accommodate the range of natural climate variability over their operating life spans. Relatively recent evidence in a large body of climate change related scientific literature shows a shifting climatological baseline about which climate variability occurs, and a changing range of that variability as well. This is relevant to USACE because the assumptions of stationary climatic baselines and a fixed range of natural variability, as captured in the historic hydrologic record, may no longer be appropriate for long-term risk projections.

Long-term, natural or anthropogenic driven climate change alter regional thermal, hydrologic, and ecological patterns. The purpose of this analysis is to qualitatively assess climate change impacts on the proposed Mill Creek General Investigation (GI). Given proposed activity in the Mill Creek GI, flood risk reduction and ecosystem restoration in the Mill Creek watershed are the main considerations in this qualitative analysis. This study also seeks to provide qualitative assessment of how hydrologic variables have responded to climate change in the past and may respond in the future. The results of this qualitative assessment may be used to increase the resilience of existing and proposed water resources projects in the watershed.

Climate change impacts on Mill Creek hydrology were considered in accordance to USACE Engineering Construction Bulletin (ECB) 2018-14, *Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs and Projects (USACE, 2018)*, as well as USACE Engineering Technical Letter (ETL) 1100-2-3 *Guidance for Detection of Nonstationarities in Annual Maximum Discharges (USACE, 2017)*. The ECB describes a qualitative analysis. The goal of such an analysis is to describe relevant observed historic and possible future climate vulnerabilities and impacts of climate change.

Mill Creek is in region 17, the Pacific Northwest Region, Middle Columbia sub region and accounting unit, and Mill Creek cataloging unit in HUC10 watershed 1707010202. Mill Creek is 76,051 acres and extents up from Walla Walla, WA to its headwaters in the Blue Mountains.

2. Existing Conditions

A brief analysis of existing conditions was conducted for the project region and site. The analysis included a literature review of regional climate behavior, identification of relevant climate variables, and analysis of observed Mill Creek flow magnitude trends.

2.1. Literature Review: Current Climate and Climate Changes Observed in the Project Area

Included in this section are points from a few publications regarding observed natural and anthropogenic climate trends relevant to the Pacific Northwest.

2.1.1. <u>Fragmented patterns of flood change across the United States (Archfield, et al 2016)</u>

This paper emphasizes a lack of cohesive pattern between catchments, indicating exceptions to regional trends and inability to predict future flooding behavior for a specific catchment based on more general climate change patterns. Authors assert that future flood frequency and magnitude changes due to enhanced greenhouse forcing are not currently generally evident over large portions of the United States for several different measures of flood flows. Even within a given region of the nation, the changes exhibited can be very different in watersheds that are in close proximity to each

other. The complex, fragmented pattern of flood flow change across the U.S. obscures generalizations.

2.1.2. Regional Climate Trends and Scenarios for the U.S. National Climate Assessment. Part 6. Climate of the Northwest U.S., NOAA Technical Report (Kunkel et al 2013)

Temperature: In the Northwest, temperatures have generally been above the 1901 – 1960 average for the last 25 years, both annually and for all seasons. Freeze-free season lengths during 1991 – 2010 averaged about 11 days longer than during 1961 – 1990. Since 1990 freeze temperatures have been occurring later in fall and not occurring earlier in spring.

Annual precipitation has exhibited very high variability since 1976. Recent years have been wet, but the majority have been below the 1901- 1960 average. Seasonal averages show similar variability, with winter having notably high variability.

Occurrence of heat waves in the Northwest has been high in recent years, with five of the top 10 years for intense heat occurring in the last two decades. Cold waves have been generally more infrequent since 1990 with all of the top ten years for intense cold occurring prior to 1991.

Although there was a period of high numbers of extreme events in the 1990s, from data considered in this study there is no evidence of an overall increasing or decreasing trend in the number of extreme precipitation events per year for the twentieth century in the Northwest US.

2.1.3. Analysis of changes in the magnitude, frequency, and seasonality of heavy precipitation over the contiguous USA (Mallakpour and Villarini 2017)

Results of this study show a stronger signal of change in frequency rather than in magnitude of heavy precipitation events in the contiguous USA. Over the last 65 years preceding 2017, the stronger storms are not getting stronger, but a larger number of heavy precipitation events have been observed.

2.1.4. Update to data originally published in: Mote, P. W., A. F. Hamlet, M. P. Clark, and D. P. Lettenmaier. 2005. "Declining Mountain Snowpack in Western North America." (Mote and Sharp 2016)

Winter and spring temperatures have increased in western North America during the 20th century, producing changes in hydrology and plants (earlier leafing and blooming). It follows that spring snowmelt has shifted earlier in the year. April 1 snowpack has decreased at most locations in the West.

2.1.5. Extreme precipitation over the west coast of North America: Is there a trend? (Mass et al 2011)

Heavy precipitation events from 1950 – 2009 were considered and found a substantial increase in major events over Washington. Maximum river discharge trends parallel that found for heavy precipitation, with increases in northern Oregon and Washington and declines in most of Oregon and northern California.

2.1.6. Climate and Hydrology Datasets for RMJOC Long-Term Planning Studies: Second Edition (RMJOC-II) Part I: Hydroclimate Projections and Analyses (Pytlak et al 2018)

This paper presents work from the River Management Joint Operating Committee (RMJOC) which includes Bonneville Power Administration, USACE, and the US Bureau of Reclamation. Authors considered data for the Columbia River Basin. Mainly the study deals with a new (at the time) set of

naturalized streamflow datasets derived from the CMIP-5 Global Model and projections of future conditions. However, the study does note that temperatures have warmed about 1.5°F since the 1970s.

2.1.7. Literature synthesis on climate change implications for water and environmental resources (Spears et al 2013)

In the 20th century all areas of the Pacific Northwest region became warmer, and some areas received more winter precipitation. Spring temperatures increased 1 to 3°C between 1970 and 1998. The Pacific Northwest experienced a general decline in spring snowpack, reduced snowfall to winter precipitation ratios, and earlier snowmelt runoff between the mid and late 20th century. In the Western US from 1950 – 1999 there was a general decrease in the fraction of precipitation retained in the spring snowpack. Snow cover extent in North America has set record lows in 3 of the 5 years preceding 2012. A study by Kapnick and Hall (2010) found that recent snowpack changes are due to regional-scale warming, which implies a possible future loss of late season snowpack and an earlier melt season. Multiple studies estimated 1°C warmer climate results in 14.8 – 20% decrease in snow water equivalence (SWE).

Runoff shows earlier peaks at most stations between 1950 and 1999, with significant trends toward earlier runoff in the Pacific Northwest. Stewart et al (2005) found that the center of mass of streamflow has shifted earlier by 1 to 4 weeks in many of the records. Other studies found runoff earlier by 1 – 3 weeks over the majority of the Mountain West. On the other hand, Fritze et al., 2011 report generally earlier snowmelt runoff, but most coastal rain-dominated and some interior basins have experienced a later timing. This study states warmer temperatures in snowpack dependent watersheds cause reduced snowpack during winter, more winter month runoff, and earlier spring peak flows associated with an earlier snowmelt so lower elevation reservoirs do not receive inflow as they did.

The former U.S. Climate Change Science Program issued a product in 2008 reporting that heavy precipitation events averaged over North America have increased over the past 50 yr. Frequency of extreme precipitation from 1948 – 2006 was reported to increase. This document further relates that Kunkel (2003) found an increase in extreme precipitation event frequency since the 1920s/1930s in the U.S. However, as can be seen in section 2.1.2 above, the same author was primary on a contradicting paper in 2013.

2.1.8. Recent US Climate Change and Hydrology Literature Applicable to US Army Corps of Engineers Missions (USACE, 2015)

This report is one of 21 regional climate reports prepared as a literature review of publications relevant to each of the 21 HUC2 watersheds in the contiguous U.S. The area covered by the Region 17 report is shown in Figure 1.

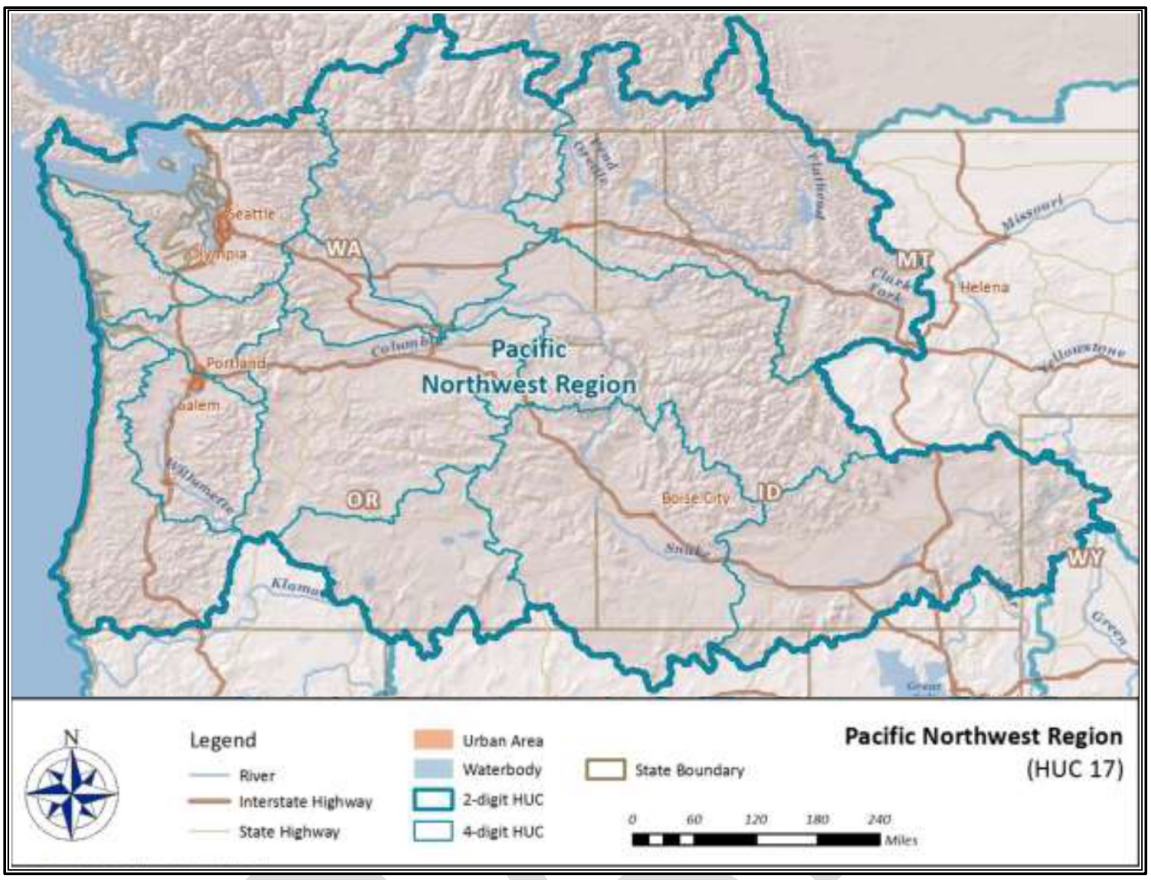


Figure 1: Water Resources Region 17: Pacific Northwest Region Boundary

Temperature: Increasing average, minimum and maximum temperatures in the 20th century. MacDonald (2010) notes no observed temperature difference between 1895 – 2000 average and average annual temperatures over 2001 - 2009 for southeastern Washington (MacDonald 2010).

Precipitation: Moderate consensus supporting increasing annual average precipitation trends in the pacific northwest. However, some studies reviewed in this document indicate a slight precipitation decrease in southeastern Washington. Soil moisture was found to be increasing in western Pacific Northwest Region based on annual data from 1895 – 2006, but decreasing slightly in the eastern portion of the Pacific Northwest Region (Grundstein, 2009). Soil moisture is a function of both supply (precipitation) and demand (evapotrasporation).

Stream Flow: This study reports a strong consensus of statistically significant decrease in streamflow and April 1 SWE data for the latter half of the 20th century.

2.1.9. Temperature changes in the United States. In: Climate Science Special Report: Fourth National Climate Assessment, Volume I (Vose et al 2017)

Temperatures from 1895 – 2011 averaged warming of about 1.3 °F. The average present-day (1986 – 2016) observed coldest daily temperature for the Northwest is 4.78°F warmer than the average for the first half of the last century (1901 – 1960). The warmest day of this same comparison is 0.17°F cooler. Temperature extremes across the contiguous US has changed. The frequency of cold waves has decreased since early 1900s, and heat wave frequency has increased since mid-1960s. The

number of high temperature records set in past two decades far exceeds the number of low temperature records.

Precipitation has generally increased, though both increasing and decreasing trends are observed among various locations, seasons, and time periods of analysis. Extreme precipitation has not significantly changed in this time period.

2.1.10. Literature Review Summary

The general consensus presented in this paper is an increased temperature in the 20th with associated increase in freeze-free season lengths and earlier snowmelt runoff as well as decreased snowpack or April 1 SWE April 1. One study also noted rising water temperatures. Soil moisture content and streamflow is noted decreasing in the project area since about 1950. Some studies indicate increasing major or extreme precipitation events, other studies assert no substantial precipitation increase.

2.2. Identification of Relevant Climate Variables

The important hydrologic variables affecting the project include air temperature, water surface elevation (stage), discharge, and hydrograph timing. Air temperature is connected to more frequent extreme precipitation events. Increased precipitation increases the other three variables.

Analyses in this study can be frustrated by a historical record at the project site that includes long-term geomorphic change or gage relocation. Stream flow magnitude can be influenced by changes in land-use, channel realignment, and measurement techniques. These factors make it difficult to determine the role of climate change at the project. The relevant question to answer at the project scale is whether there has been, or will be, a change that affects conditions in the study area and how this change would impact the resilience of the proposed project. Discharge was the hydrologic variable to analyze for this project.

Relevant components of stream discharge include timing, magnitude, frequency, duration, and associated stages. An increase in magnitude may result in higher water surface elevations contributing to levee or dam overtopping and inundation of protected areas. More frequent or longer duration flood conditions can stress levee systems or an earthen dam. Bank-full or channel forming flow may carry bedload or increased suspended sediments into the off-channel storage and counteract excavation efforts to increase flood storage capacity.

2.3. Site Specific Analysis: Trends in Observed Streamflow Records

This portion of the climate change assessment focuses on carrying out first order statistical analysis using stream flow records observed at the USGS gauge Mill Creek at Walla Walla (gauge 14015000) and annual peak data utilized in the Mill Creek General Investigation (GI).

2.3.1. Nonstationarity and Trend Analyses

The project area, shown in Figure 2 below, is located within the Mill Creek Watershed near Walla Walla, WA. Three active USGS gauges are placed along Mill Creek at or upstream of the project area. The data used in this analysis come from the gauge indicated in Figure 2. Details for this gauge are summarized in Table 1.

Table 1. Summary of USGS Gage details used for Analysis

USGS Stream Gage	Period Of Record	Drainage Area (mi^2)
14015000 – Mill Creek at Walla Walla, WA	1942 – 1948, 1950 – 2019	95.7

Figure 2. Mill Creek watershed at the HUC-10 level. Study area is near the USGS gauge 14015000.

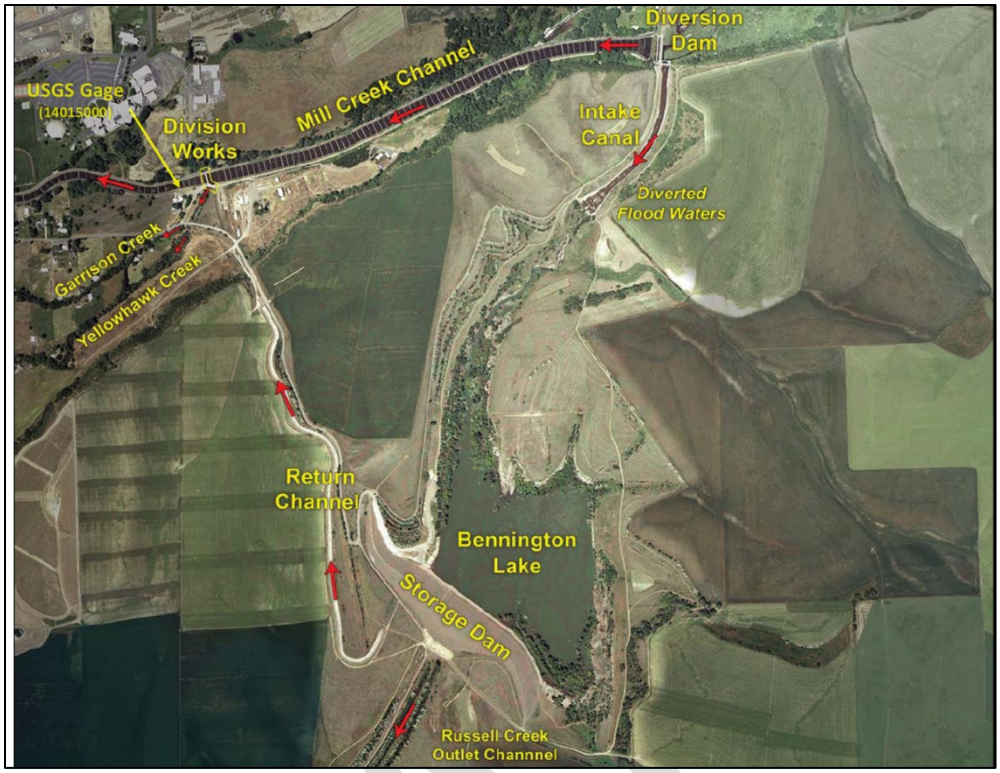


Figure 3. Location of the project area with typical water flow paths indicated. Bennington Lake is outside the HUC-10 boundary that includes Mill Creek channel and the USGS gauge used in this study.

The USACE Nonstationarity Detection Tool was applied to determine if flows recorded at the project indicated stationary conditions. This tool was applied to both USGS data directly accessed through the tool as well as an annual peak time series developed from hourly peaks that were used in the Mill Creek GI.

The nonstationarity tool applies 12 statistical tests to detect trends in the data, five of which are change point models, a Bayesian change point, smooth and abrupt Lombard methods, the energy divisive method, and the Pettitt test. Default sensitivity parameters (Table 2) for the tests were initially applied, then the sensitivity parameters were varied for each data set. The Mill Creek at Walla Walla gauge maximum peak occurred in the flood of 1996, which caused extensive damage in Walla Walla. The starting date for this nonstationarity analysis was moved from 1942 up to 1950 because 1949 was missing from the data set, specifying a continuous period of record from 1950 to the present year (2019) for this analysis.

Table 2. Default, Minimum, and Maximum Nonstationarity Detection Tool Sensitivity Parameters

Sensitivity Parameters	Default	Min	Max
Cramer-Von-Mises Methods Burn-In Period	20	5	50
CPM Methods Sensitivity	1,000	370	50,000

Bayesian Posterior Threshold	0.5	0	1
Energy Divisive Method Sensitivity	0.5	0.1	0.9
Lombard Smooth Methods Sensitivity	0.05	0.01	0.1
Pettitt Sensitivity	0.05	0.01	0.1

As can be seen in Figure 4, performing a nonstationarity test on the USGS data directly reveals no nonstationarities. This gauge is below two diversions: one feeding the off-channel flood storage reservoir Bennington Lake, and the other a diversion into Yellowhawk and Garrison Creeks. Thus, this data is regulated. The data set can be considered stationary over the entire continuous period of record. The associated tool output heat map showing a graphical representation of the statistical results is not included in this report because it was blank due to lack of nonstationarities. The segmented mean, segment standard deviation, and segment variance output from the nonstationarity tool are constants at 1,121 cfs, 617.8 cfs, and 381,665 cfs^2, respectively. Sensitivity parameters were adjusted to highest sensitivity for all parameters listed above and nonstationarities still did not result.

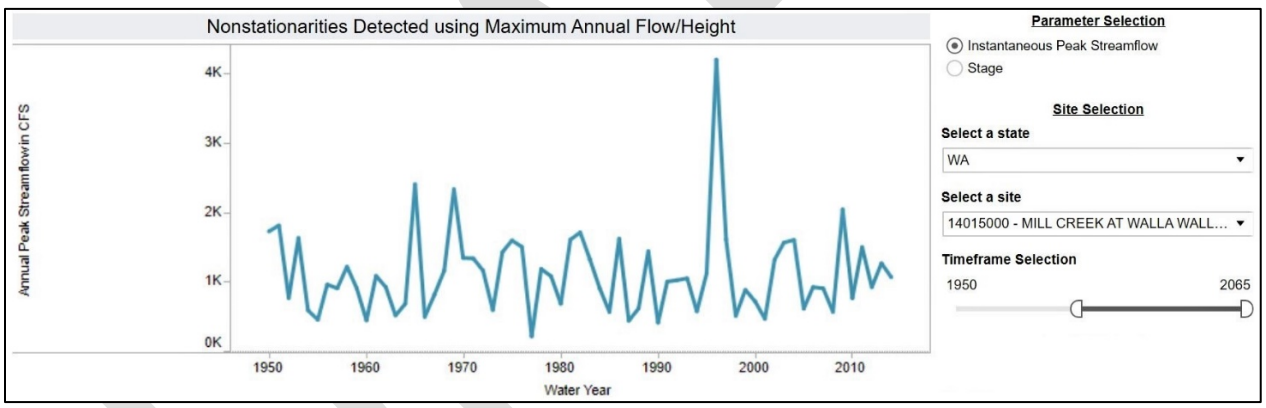
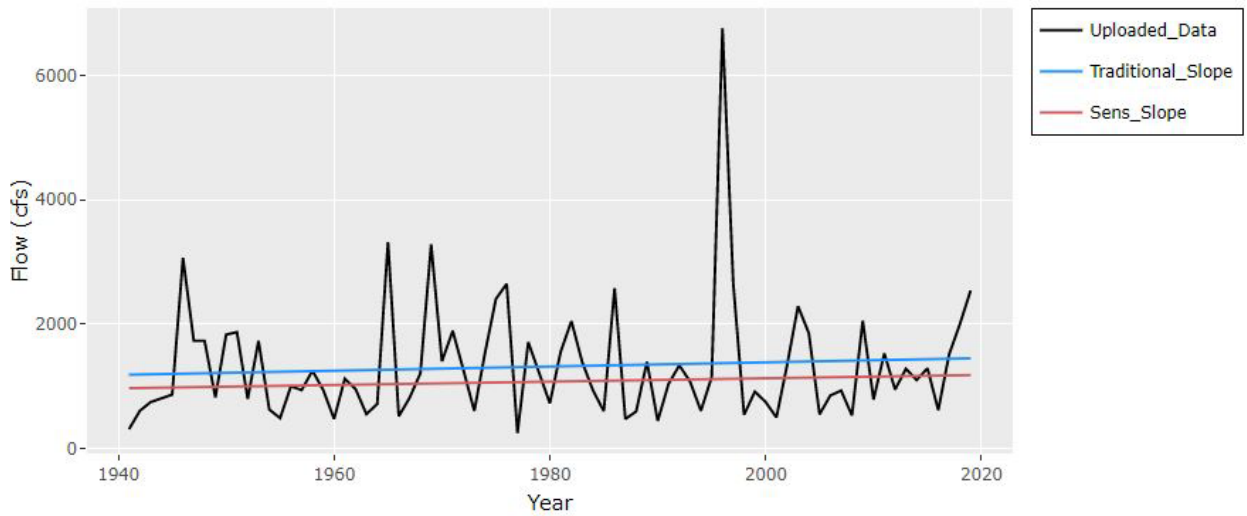


Figure 4. Nonstationarity Detection Analysis of Maximum Annual Flow, USGS Gage 14015000 – Mill Creek at Walla Walla

A separate analysis was performed for the unregulated data used by the Mill Cr GI product development team (PDT). The data was unregulated by the PDT through a series of steps that will not be detailed in this report and the time Series Toolbox trend analysis was performed as shown in Figure 5.



Trend Line Coefficients

Method	Slope	Intercept
Linear Regression Slope	3.3752	-5366.6
Sens Slope	2.6939	-4260.9

Figure 5 Hourly annual peak data with slope trend lines and coefficients

Analysis of the time series reveals no significant trend at the alpha = 0.05 level. P-values for the t-Test, Mann-Kendall, and Spearman Rank-Order tests are 0.47, 0.41, and 0.43 respectively. This indicates no significant trend in the data, despite the fact that trend line coefficients in Figure 5 were not zero. Regression techniques used to fit the trend lines to the data outputs summary statistics characterizing the time series presented in Table 3 Data characterization summary statistics for hourly annual peak data used in the Mill Creek GI.

Table 3 Data characterization summary statistics for hourly annual peak data used in the Mill Creek GI

Statistic	Value
Mean (cfs)	1316.3
Coefficient of Variation	0.34
Skewness	0.32
Kurtosis	0.2
AR1	0.1

The time series of maximum annual hourly peak flows used for the Mill Creek GI was uploaded to the USACE Time Series Toolbox and nonstationarity analysis was also performed on this data.

Results are shown in Figure 6. Statistical test name acronyms are: Cramer-von-Mises (CVM), Kolmogorov-Smirnov (KS), LaPage (LP), Energy Divisive (END), Lombard Wilcoxon abrupt (LW), Pettitt (PT), Mann-Whitney (MW), Bayesian Change Point (BAY), Lombard Mood (LM), Mood (MD), Smooth Lombard Mood (SLM), and Smooth Lombard Wilcoxian (SLW). All tests except the SLW nonstationarity detection method show stationary data. The USACE Nonstationarity Detection Tools User Guide (Version 1.2 updated 2018) indicates that the Lombard Wilcoxon test does not work well with short time series or with small changes in magnitude. Parameter adjustments did not change the results for any of the tests. Given the fact that the SLW tests for smooth changes in statistical properties of the dataset, such as mean and variance, and the nonstationarity registered at the start of the time series, this nonstationarity is likely due to the lack of preceding data rather than an actual statistical mean or variance change if the data extended continuously prior to 1942. Therefore, this nonstationarity is disregarded and the hourly daily maximum data is assumed to be stationary. This agrees with the nonstationarity and trend analysis of the data directly and automatically pulled from the USGS gauge and presented above. The test parameters were identical to the parameters presented in Table 2 except that Bayesion Prior Likelihood was added to the list with a default value of 0.2.

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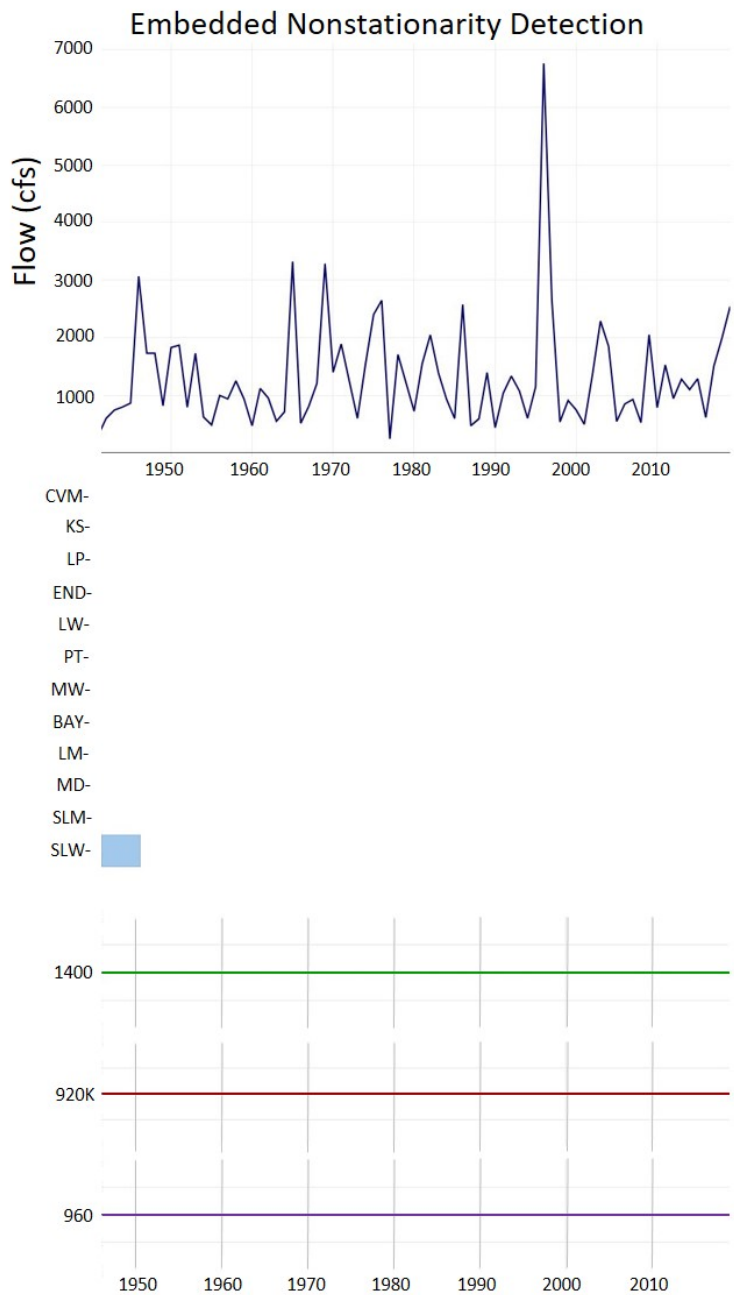

Figure 6 Time series tool nonstationarity detection statistical tests.

Breakpoint analysis was also performed in the Time Series Toolbox, but no breakpoints were detected. Furthermore, analysis was performed on the time series to determine an appropriate time series model based on the seasonality, trends, and nonstationarities manifest in the data. The resulting visualization is presented in Figure 7.

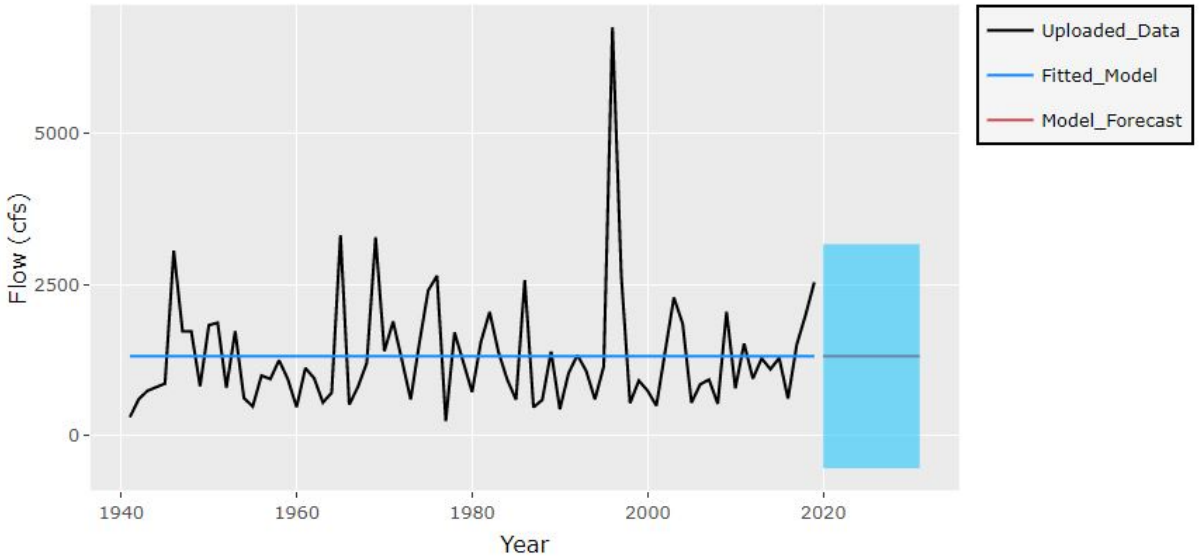

Figure 7 Auto Regressive Integrated Moving Average model.

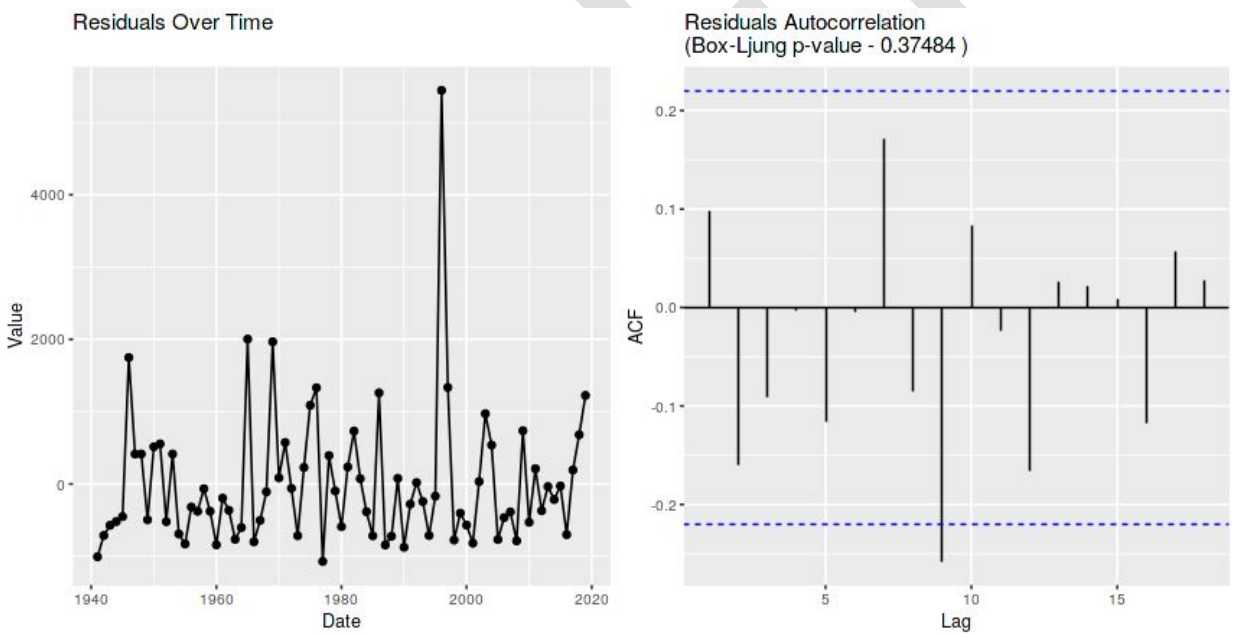

Figure 8 ARIMA residuals from time series analysis.

For comparison, the Exponential Smoothing time series model was also applied to the data set. The outcome (not shown) was nearly identical, except that the trend line shifted from a horizontal line at 1316.266 cfs with ARIMA to a slightly sloping line with ETS moving from (year, flow cfs) of (1941, 1248.182) to (2019, 1248.590).

Land use data was reviewed for possible impacts on flow by viewing historical aerial photos of the watershed. In the last 25 years simple visual inspection reveals no significant land use change. Some localized development along the stream has occurred, but apparently nothing significant.

3. Future Without-Project Condition

3.1. Literature Review: Projected Climate Change and Projected Changes in Climate Variables

3.1.1. Implications of 21st century climate change for the hydrology of Washington State (Elsner et al 2010)

This study notes that Pacific Northwest hydrology is sensitive to temperature change impact on rain/snow balance because of dependency on snowmelt. April 1 snow water equivalence (SWE) is predicted to decrease by 38 – 46% by the 2040s compared with the mean over water years 1917 – 2006. By the 2080s, seasonal streamflow timing will shift in both snowmelt dominated and rain-snow mixed watersheds. Annual runoff across the state is projected to increase by 2 – 3% by the 2040s given an increase in winter precipitation

3.1.2. Regional Climate Trends and Scenarios for the U.S. National Climate Assessment. Part 6. Climate of the Northwest U.S., NOAA Technical Report (Kunkel et al 2013)

Temperature: Increases in temperature inland are greater than those near the coast. The freeze-free period increases by 25 – 35 days across much of the Northwest region, with larger increases west of the Cascade Mountains. This study also predicts an increase in the number of days hotter than 95°F in the southeast portion of the region. The longest string of days with such high temperatures is simulated to increase by up to 10 days per year.

Precipitation: Seasonal mean precipitation increase is simulated for southeast Washington except for winter which is projected to experience a 2 – 4 % decrease. However, there is great uncertainty associated with precipitation changes. The number of wet days (precipitation > 1 in) is forecasted to increase, but changes are statistically significant only for small areas in central Washington and Oregon.

3.1.1. Climate change impacts on streamflow extremes and summertime stream temperature and their possible consequences for freshwater salmon habitat in Washington State (Mantua et al 2010)

Rising water temperatures will thermally stress salmon throughout Washington watersheds, becoming increasingly severe later in the twenty-first century. Basins strongly influenced by transient runoff (a mix of direct runoff from cold-season rainfall and springtime snowmelt) are most sensitive to climate change. By the 2080s, hydrologic simulations predict a complete loss of Washington snowmelt dominant basins, and only about ten transient basins remaining in the north Cascades. Historically transient runoff watersheds will shift toward rainfall dominant behavior, with more severe summer low flow periods and more frequent days with intense winter flooding. Winter and spring warming may benefit parts of the freshwater life-cycle of some salmon populations, but the combined effects of warming summertime stream temperatures and altered stream flows will likely strain many salmon populations. Increased winter flooding in transient runoff watersheds will likely reduce the egg-to-fry survival rates. The quality and extent of freshwater salmon habitat will likely reduce due to rising stream temperatures.

3.1.2. Extreme precipitation over the west coast of North America: Is there a trend? (Mass et al 2011)

When this study's geographical pattern of extreme West Coast precipitation was compared to other literature, the authors cite both agreeing and disagreeing studies that show a decrease, no trend, and increasing prediction over the Pacific Northwest. In closing the authors cite another study stating that natural variability may be the cause of observed extreme precipitation patterns.

3.1.3. Assessing regional impacts and adaptation strategies for climate change: the Washington Climate Change Impacts Assessment (Miles et al 2010)

Water availability is highly variable and is expected to change as the climate warms. Summer and fall low flow season will substantially increase in length, exacerbating direct effects of warmer air temperatures on stream temperature. Rising stream temperatures will reduce freshwater salmon habitat. Basins will be increasingly rain dominated by mid-century. Further, this study predicts temperature increases of 2 – 3°F by the 2040s. Based on their analysis, they project greater western and far eastern Washington precipitation but less precipitation in the lower Columbia basin. Like most other studies reviewed, this study projects decreasing April 1 SWE.

3.1.4. Update to data originally published in: Mote, P. W., A. F. Hamlet, M. P. Clark, and D. P. Lettenmaier. 2005. "Declining Mountain Snowpack in Western North America." (Mote and Sharp 2016)

Temperature increase estimates in the Western US range from 2° - 5° C over the next century, but changes in precipitation are inconsistent and average near zero. Given temperature increases it is likely that losses in snowpack observed up to 2005 will likely continue and even accelerate, with faster losses in milder climates like much of the Cascades and the slowest losses in the higher altitude Rockies and Sierra.

3.1.5.Future climate in the Pacific Northwest (Mote and Salathé 2010)

Climate models used in the Intergovernmental Panel on Climate Change (IPCC) Fourth Assessment Report (AR4) generally reproduce the observed seasonal cycle and twentieth century warming trend of 0.8°C (1.5°F) in the Pacific Northwest, and point to much greater warming for the next century. These models project increases in annual temperature of, on average, 1.1°C (2.0°F) by the 2020s, 1.8°C (3.2°F) by the 2040s, and 3.0°C (5.3°F) by the 2080s, compared with the average from 1970 to 1999, averaged across all climate models. Rates of warming range from 0.1°C to 0.6°C (0.2°F to 1.0°F) per decade. Projected changes in annual precipitation averaged over all models are small (+1% to +2%), but some models project an enhanced seasonal cycle with changes toward wetter autumns and winters but drier summers.

3.1.1. Climate and Hydrology Datasets for RMJOC Long-Term Planning Studies: Second Edition (RMJOC-II) Part I: Hydroclimate Projections and Analyses (Pytlak et al 2018)

Temperatures are expected to warm another 1 to 4°F by the 2030s. Warming is likely to be greatest in the interior and less near the coast. Future precipitation trends are uncertain, but a general upward trend is likely for the rest of the 21st century, particularly in the winter months. Summer months could become drier. Winter snowpack is very likely to decline as more winter precipitation falls as rain. By the 2030s this study predicts higher average fall and winter river flows, earlier peak spring runoff, and longer periods of low summer flows as very likely.

3.1.2. Precipitation extremes and the impacts of climate change on storm water infrastructure in Washington State (Rosenberg et al 2010)

Downscaled regional climate model based on the ECHAM5 and CCSM3 global climate models indicate a possible increase in extreme rainfall magnitudes, though the range of the projections is too large to provide a basis for engineering design and needs a larger sample of simulated climate data. Still, they suggest that drainage infrastructure designed using mid-20th century rainfall records may be subject to a future rainfall regime that differs from current design standards.

3.1.3. Regional climate model projections for the State of Washington (Salathé et al 2010)

Because global climate models do not have sufficient spatial resolution to represent the atmospheric and land surface processes comprising unique regional climate of the State of Washington, this regional climate model study is very relevant. Large-scale weather patterns simulated by a global model interact with the local terrain. This study reports two 100-year regional climate simulations. The mesoscale simulations produce regional changes in snow cover, cloudiness, and circulation patterns, which affect temperature and precipitation trends over the region relative to the global model or to statistical downscaling. To illustrate this effect, this study analyzes the changes from the current climate (1970–1999) to the mid twenty first century (2030–2059). Main findings from this paper are 1. Projected loss of snowpack, 2. Reduced snowpack and earlier snowmelt will alter timing and amount of river runoff in the summer, though changes in annual runoff will depend on annual precipitation changes which differ from one scenario to another, and 3. Extreme precipitation frequency increases over the north Cascades and over eastern Washington.

3.1.4. Literature synthesis on climate change implications for water and environmental resources (Spears et al 2013)

Analyses suggested both losses in snowpack in lower altitude mountain ranges (Bales et al. 2006) and high altitude or high latitude cool-season increase during the 21st century. Pierce and Cayan (2012) report projected shift from snow to rain predominantly in the Western US. Studies suggest increased precipitation in the northern tier of the Western US could counteract warming-related decreases in warm season runoff, while decreased precipitation in southern states would amplify decreased warm season runoff decreases. Uncertainty is very large for precipitation projections for many parts of the US over the next 10 to 60 years.

On the Columbia River at The Dalles near annual temperature increases 4.6 °F by the 2070s, mean annual precipitation increases 8.5%, mean annual runoff increases 7.5%, mean December – March runoff increases 27.3%. These predictions are summarized in Table 1 (on p. 14 of the document and below) with similar predictions for the Snake River at Brownlee Dam and the Yakima River at Parker also presented.

Table 1.—Summary of simulated changes in decade-mean hydroclimate for several subbasins in the Columbia River Basin from an ensemble of downscaled CMIP3 models run through VIC

Hydroclimate Metric (Change from 1990s)	2020s	2050s	2070s
Columbia River at The Dalles			
Mean Annual Temperature (°F)	1.4	3.2	4.6
Mean Annual Precipitation (%)	3.4	6.2	8.5
Mean April 1 SWE (%)[1]	-1.0	-3.1	-6.7
Mean Annual Runoff (%)	2.3	3.7	7.5
Mean December–March Runoff (%)	9.8	18.5	27.3
Mean April–July Runoff (%)	2.2	4.1	2.4
Mean Annual Maximum Week Runoff (%)	3.5	4.0	5.5
Mean Annual Minimum Week Runoff (%)	-1.5	-5.9	-8.5
Snake River at Brownlee Dam			
Mean Annual Temperature (°F)	1.6	3.6	5.0
Mean Annual Precipitation (%)	2.3	3.9	6.6

Mean April 1 SWE (%)[1]	-5.0	-12.0	-16.0
Mean Annual Runoff (%)	-0.1	1.2	3.4
Mean December–March Runoff (%)	5.6	13.7	21.0
Mean April–July Runoff (%)	-1.3	-2.0	-0.9
Mean Annual Maximum Week Runoff (%)	2.4	3.5	5.8
Mean Annual Minimum Week Runoff (%)	-3.0	-4.3	-5.9
Yakima River at Parker			
Mean Annual Temperature (°F)	1.3	2.9	4.2
Mean Annual Precipitation (%)	3.7	5.7	7.7
Mean April 1 SWE (%)[1]	-10.3	-19.6	-28.7
Mean Annual Runoff (%)	3.8	3.7	5.6
Mean December–March Runoff (%)	19.6	39.9	56.9
Mean April–July Runoff (%)	-2.0	-9.5	-17.0
Mean Annual Maximum Week Runoff (%)	2.7	4.2	6.7
Mean Annual Minimum Week Runoff (%)	-4.0	-10.6	-14.2

[1]The reported percentage changes in mean April 1 SWE have been updated to correct a reporting error in Reclamation (2011b). The error stemmed from reporting this change as the mean change in cell-specific changes from all 1/8-degree grid-cells within the given basin. Such a change metric does not equal the change in total basin SWE integrated across all grid-cells within the basin, which was the intended reporting metric and is now indicated by the updated percentage changes.

Climate change studies suggest that April 1 snowpack is projected to decrease by 28% across Washington by the 2020s, 40% by the 2040s, and 59% by the 2080s (relative to the 1916 – 2006 historical average). Seasonal streamflow timing will likely shift significantly in sensitive watersheds.

This literature synthesis points out a consensus among some studies that extreme precipitation event frequency will increase.

With a focus on the greater Columbia River Basin, Payne et al. (2004) evaluated reservoir operations under projected hydrologic conditions and suggested increased winter runoff may necessitate earlier winter flood control drawdown relative to current dates.

Increased temperatures stress forests and cause increased insect kill vulnerability as well as increased geographic range for the insects doing the damage. Such an ecosystem shift may increase fire susceptibility and water erosion and sediment load in streams.

Warming in the Pacific Northwest is likely to have a greater effect on stream temperatures in streams dominated by snowmelt rather than those dominated by rain. Duration of periods causing thermal stress and migration barriers to salmon is projected to increase by a factor of two or four depending on the emissions scenario by the 2080s. The region of greatest expected thermal stress includes the interior Columbia River Basin. Battin et al. (2007) notes negative salmon habitat effects of the increased mean air temperature but suggested habitat restoration could mitigate the projected negative impacts of anthropogenic impacts. Arismendi et al. (2012) found little evidence for warming stream temperatures and emphasize the need for understanding mechanisms linking stream temperature to human effects and better sensor networks.

The January 2013 Federal Advisory Committee Draft Climate Assessment Report (chapter 21, p 725) says “Some Northwest streams (Isaak et al. 2011) and lakes have already warmed, on average, over

the past three decades, contributing to changes such as earlier Columbia River sockeye salmon migration (Crozier et al. 2011) and earlier blooms of algae in Lake Washington (Winder and Schindler 2004). As species respond to climate change in diverse ways, there is a potential for ecological mismatches to occur – such as in the timing of the emergence of predators and their prey (Winder and Schindler 2004)."

Invasive species (e.g. quagga mussels) may be aided by warmer water temperatures. Effectiveness of chemical or biological agents used to control invasives could decrease. Increased water temperatures could also spur algal growth, which could result in eutrophication and changes in species composition. Wetlands may also be impacted. Although freshwater ecosystems will adapt to climate change, other stressors will come into play, e.g. land use change, acid rain, habitat degradation. Adaptation of these ecosystems to climate change will likely entail a diminished native biodiversity.

3.1.5. Recent US Climate Change and Hydrology Literature Applicable to US Army Corps of Engineer Missions – Pacific Northwest Region

Although a moderate consensus is noted that air temperatures will increase over the next century in the Pacific Northwest Region, there is a strong consensus of large (ranging from roughly 5 – 15 °F) increase in maximum temperature extremes. A strong consensus is also noted that intensity and frequency of extreme storm events will increase. Future minimum temperature, average annual precipitation, and streamflow show varied trends, which may be due to the varied topography and the current ability of global climate models to resolve topographic effects from local climate forcings. Trends are summarized in Figure 9.

Effects of changing climate are briefly discussed, e.g. increased precipitation causing increased runoff and flash floods. Flood risk management projects may need to handle more frequent and intense extreme storm events. Ecosystem restoration may play a role in future aquatic ecosystem considerations, given increased ambient air temperatures and heat wave days which will increase water temperatures. Water quality, particularly dissolved oxygen levels, will respond to the temperature increase. Mitigation would be required to maintain aquatic habitat for salmonids or bull trout that reside in or migrate into the Mill Creek watershed.

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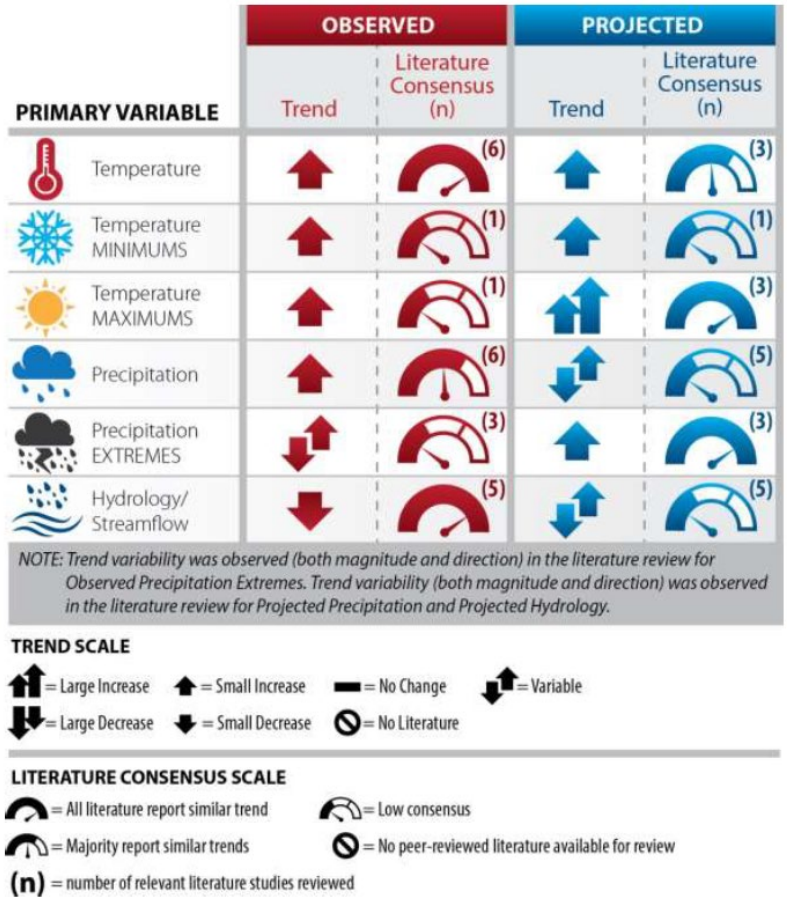

Figure 9 Summary of observed and projected climate trends and literature consensus for the Pacific Northwest.

3.1.6. Temperature changes in the United States. In: Climate Science Special Report: Fourth National Climate Assessment, Volume I (Vose et al 2017)

Temperature: Annual average temperature over the contiguous United States is projected to increase by about 2.5°F for the period 2021-2050 relative to 1976-2005. Much larger rises (2.8° - 11.9°F) are projected by late century (2071-2100). Extreme temperatures in the contiguous United States are projected to increase. Cold waves are projected to become less intense while heat waves will become more intense and below freezing days are projected to decline while the number above 90°F is projected to rise.

Stream flow: Modeling studies indicate, with near 100% likelihood for all emissions scenarios, that reductions in summer flow will occur by 2050 in basins with significant snowmelt. Mixed rain-snow watersheds will see increased winter flows, earlier spring peak flows, and decreased summer flows in a warming climate.

Precipitation: Averaged over the northwest region, the number of days with more than one inch of precipitation is projected to increase 13% in 2041 – 2070 compared with the period from 1971 – 2000 given continuation of current rising emissions trends, though these projections are not consistent across models.

3.1.7. Literature Review Summary

Temperature is projected to increase by varying amounts, but something on the order of 3.5 – 9° F over the next century. An increased freeze-free period and loss of April 1 snowpack follows from this prediction and is also discussed in the literature. Stream flows are predicted to decrease in summer, exacerbating increased temperature effect on aquatic habitat. River hydrograph timing is predicted to continue to shift earlier in the year. The combination of increasing temperature and changing hydrology are predicted to result in loss of freshwater salmon habitat and other ecological mismatches. Some literature predicts increased intensity and frequency of extreme storm events, while other studies project no trend of increasing or decreasing precipitation.

3.2. Regional Scale Analysis: Trends in Projected Streamflow Records

3.2.1. Climate Hydrology Assessment: Projected Trends in Streamflow and Climate Change at a Regional Scale

The USACE Climate Hydrology Assessment Tool was used to investigate potential future trends in stream flow for HUC 1707 shown in Figure 10 below.

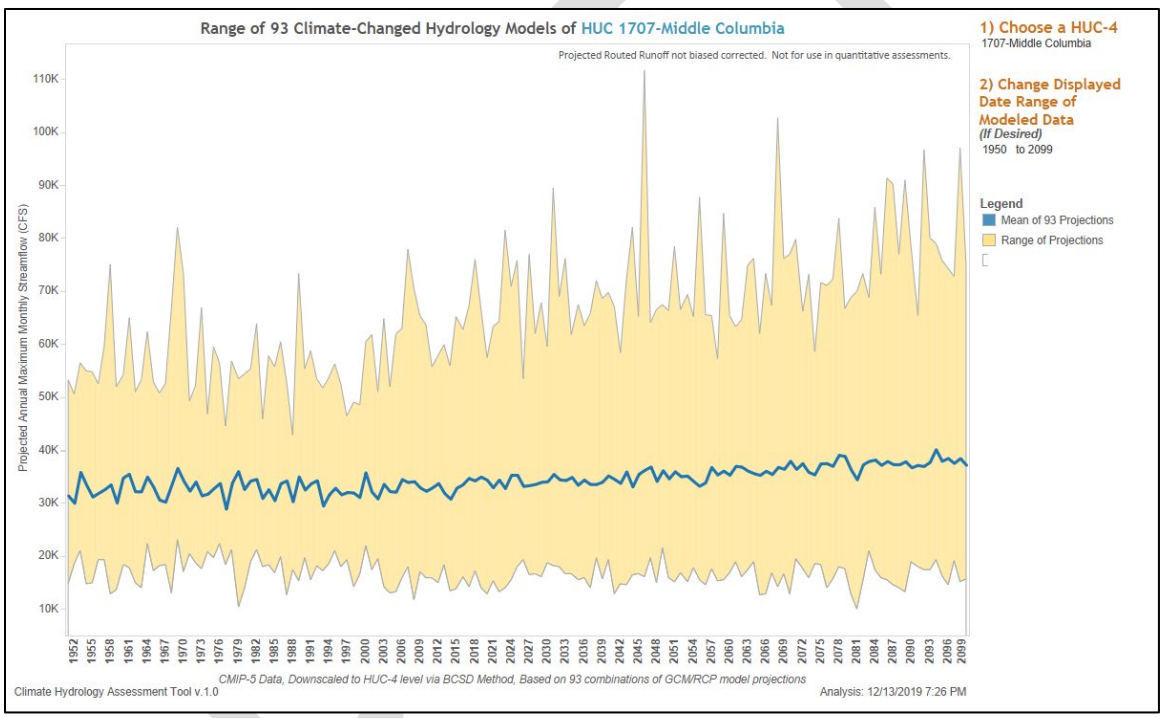

Figure 10. Mean Projected Annual Maximum Monthly Streamflow among Ensemble of 93 Climate-Changed Hydrology Models, HUC 1707 Middle Columbia

Figure 11 shows the range of projected annual maximum monthly stream flows computed from 93 different climate changed hydrologic model runs for the period of 1950-2099. Hydrology altered by climate change is generated using various greenhouse gas emission scenarios (RCPs) and global circulation models (GCM) to project precipitation and temperature data into the future. These meteorological outputs are spatially downscaled using the BCSD statistical method and then inputted in the U.S. Bureau of Reclamation's Variable Infiltration Capacity (VIC) precipitation-runoff model to generate a stream flow response. As expected for this type of analysis, there is

considerable spread in the projected annual maximum monthly flows, reflecting of model uncertainty.

The overall trend in the mean observed annual maximum monthly stream flow from the regulated USGS gauge Mill Creek at Walla Walla (14015000) decreased over time, in disagreement to the projected HUC4-level increasing trend. This shows how the larger scale trend does not accurately capture smaller scale topographic or HUC-10 level projected climate change resolution (see Rosenberg et al., 2010). These findings suggest that better modeling is necessary to determine whether or not there is increased flood risk potential relative to the current conditions. Perhaps Mill Creek has more potential for decreased flow risk and the ecological and recreation issues. It should be noted that these trends, while indicative of decreasing and increasing flows over time, are relatively small magnitude, and results here are qualitative only.

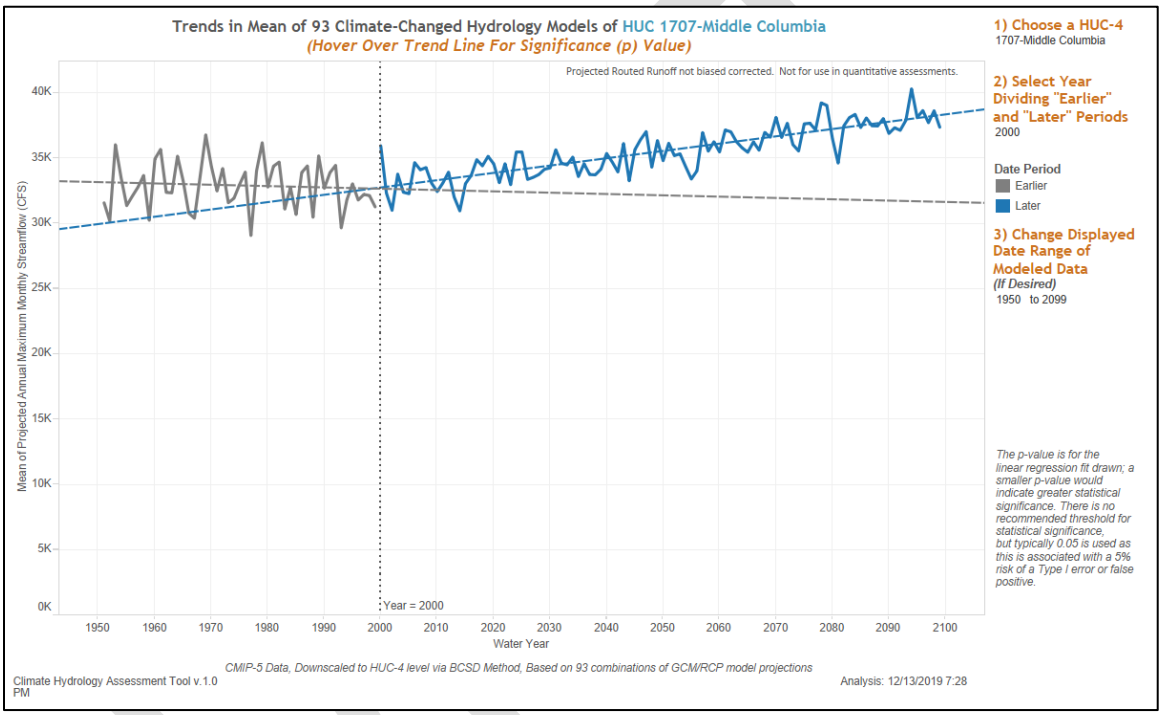

*Figure 11. Mean Projected Annual Maximum Monthly Streamflow, HUC 1707 Middle Columbia. Earlier trend line equation: Q = -10.114 * (Water Year) + 52845, p-value = 0.58. Later trend line equation: Q = 55.9316 * (Water Year) – 79157, p-value < 0.0001.*

3.2.2. Screening Level Vulnerability Assessment to Climate Change Impacts

The USACE Watershed Climate Vulnerability Assessment Tool is a screening level, comparative assessment of how vulnerable a given HUC-4 watershed is to the impacts of climate change relative to the other HUC-4 watersheds within the continental United States (CONUS). The tool can be used to assess the vulnerability of a specific USACE business line such as "Flood Risk Reduction" to projected climate change impacts. The tool uses the Weighted Order Weighted Average (WOWA) method to represent how vulnerable each business line of a HUC-4 watershed is to climate change. HUC-4 watersheds with the top 20% of WOWA scores are flagged as being vulnerable. Flood risk reduction, recreation, and ecosystem restoration were expected to be the most relevant business lines to Mill Creek.

The Climate Vulnerability Assessment Tool makes an assessment for two 30-year epochs of analysis centered at 2050 and 2085. These two periods were selected for consistency with many other national and international analyses. The Vulnerability tool uses projected climate outputs from general climate models (GCMs) and representative concentration pathways (RCPs) resulting in 100 traces per watershed per time period. The Variable Infiltration Capacity (VIC) Macroscale hydrologic model translates meteorological data projected by the GCMs into runoff. Default National Standards Settings were used in this study.

For HUC 1707 the vulnerability tool business line detail indicated zero vulnerability for any of the business lines considered (Figure 12). Only one of four scenarios (Dry – 2050, dry – 2085, wet – 2050, wet – 2085) is shown as an example but all scenarios appeared identical, registering no vulnerability in any business line.

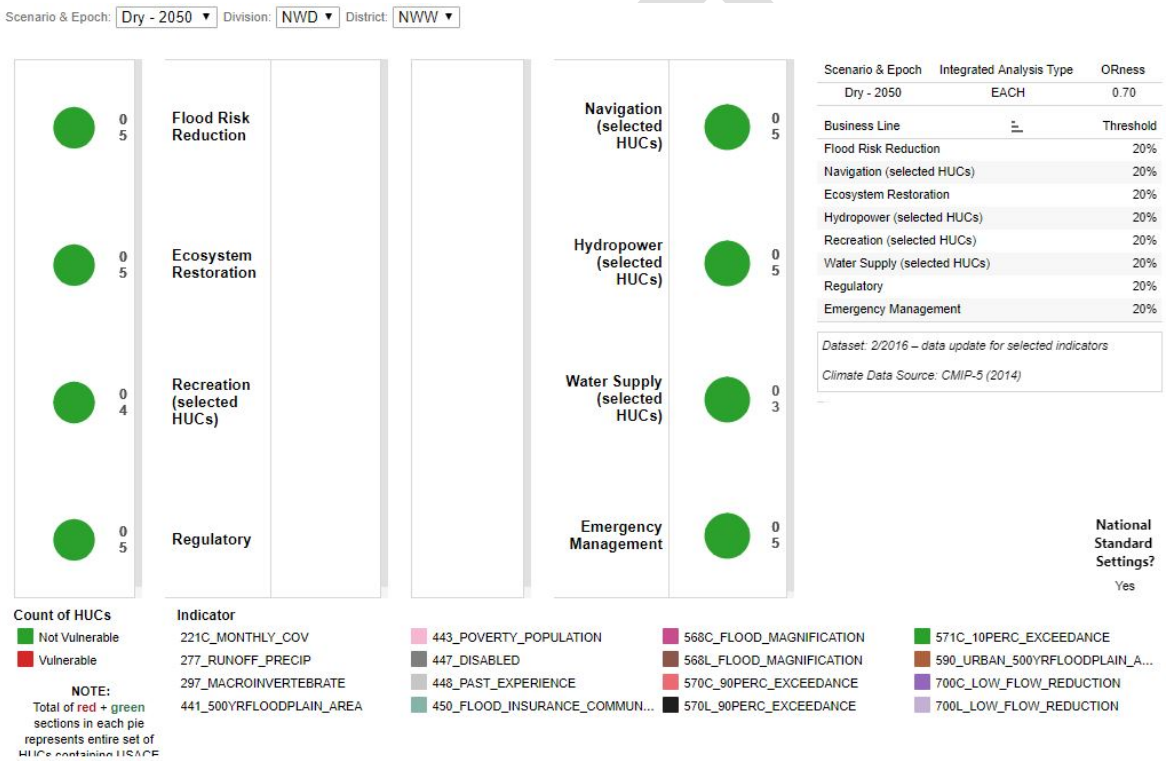

Figure 12. Vulnerability for the NWW District for all business lines

For the Ecosystem Restoration business line, under the dry scenario the Vulnerability Score for HUC 1707 changes from 64.940 in 2050 to 64.879 in 2085, and under the wet scenario it changes from 64.948 to 65.251 for the same years. For the Flood Risk Reduction business line for the dry scenario the vulnerability score was 38.299 (2050) and 38.959 (2085), and for the wet scenario 42.614 (2050) and 42.156 (2085). For the Recreation business line, the dry scenario vulnerability score was 62.361 (2050) and 63.287 (2085). Mill Creek is a water supply watershed for Walla Walla, WA, but the Water Supply business line indicated no vulnerability for Huc4 1707 so that business line is not further considered in this study. As an illustration of elements comprising business lines important to Mill Creek we include the breakdown for ecosystem restoration, Flood Risk Management, and Recreation in Figures Figure 13, 21, and 22, respectively. The dry scenario is presented, but for all of

these business lines the contributing elements of each business line followed the same pattern and exhibited similar component contributions.

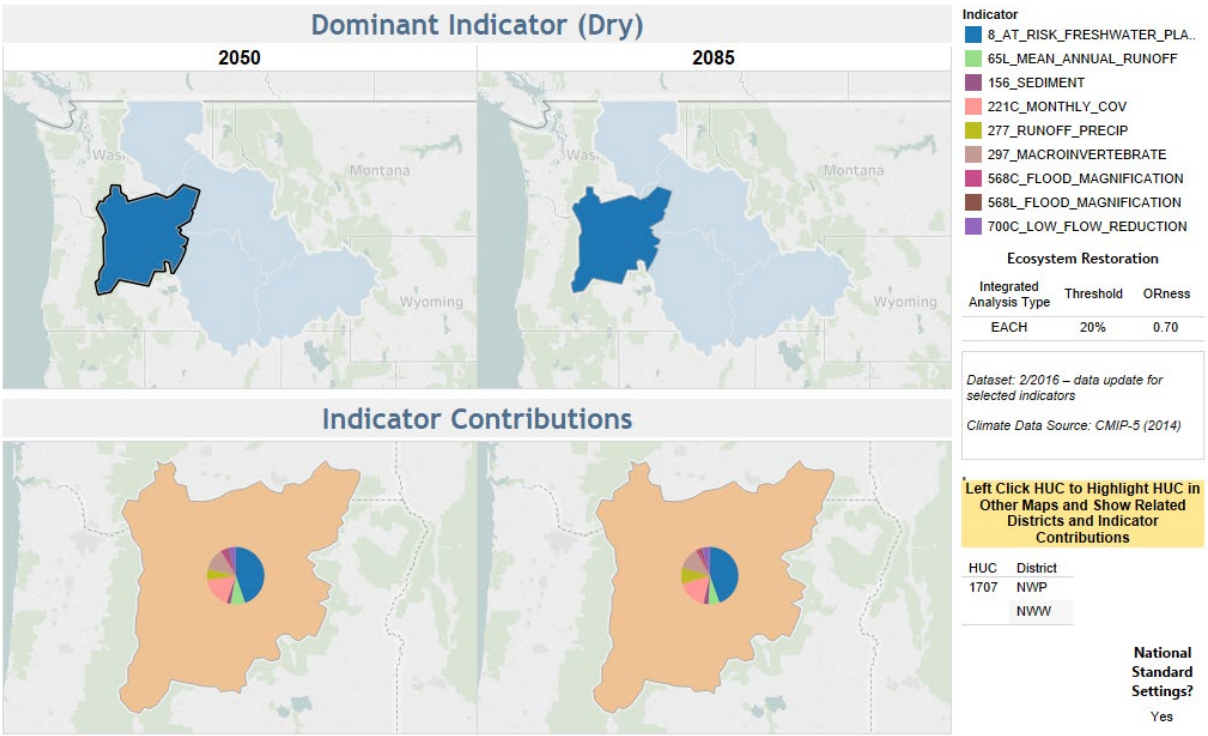

Figure 13 Dry scenario for Ecosystem Restoration. At risk freshwater plants are the dominant indicator in the WOWA score at 45% for both years

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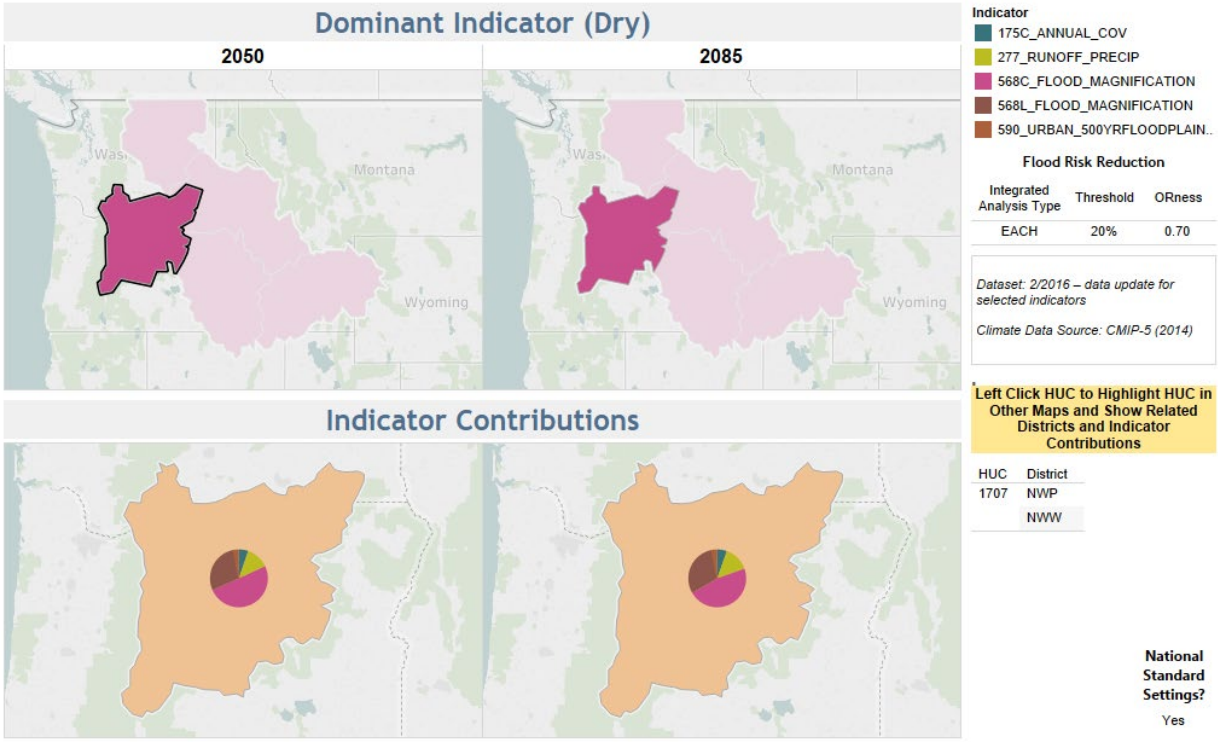

Figure 14 Dominant indicator for the Flood Risk Reduction business line is flood magnification at approximately 50% of the WOWA score for each year for dry and wet scenarios.

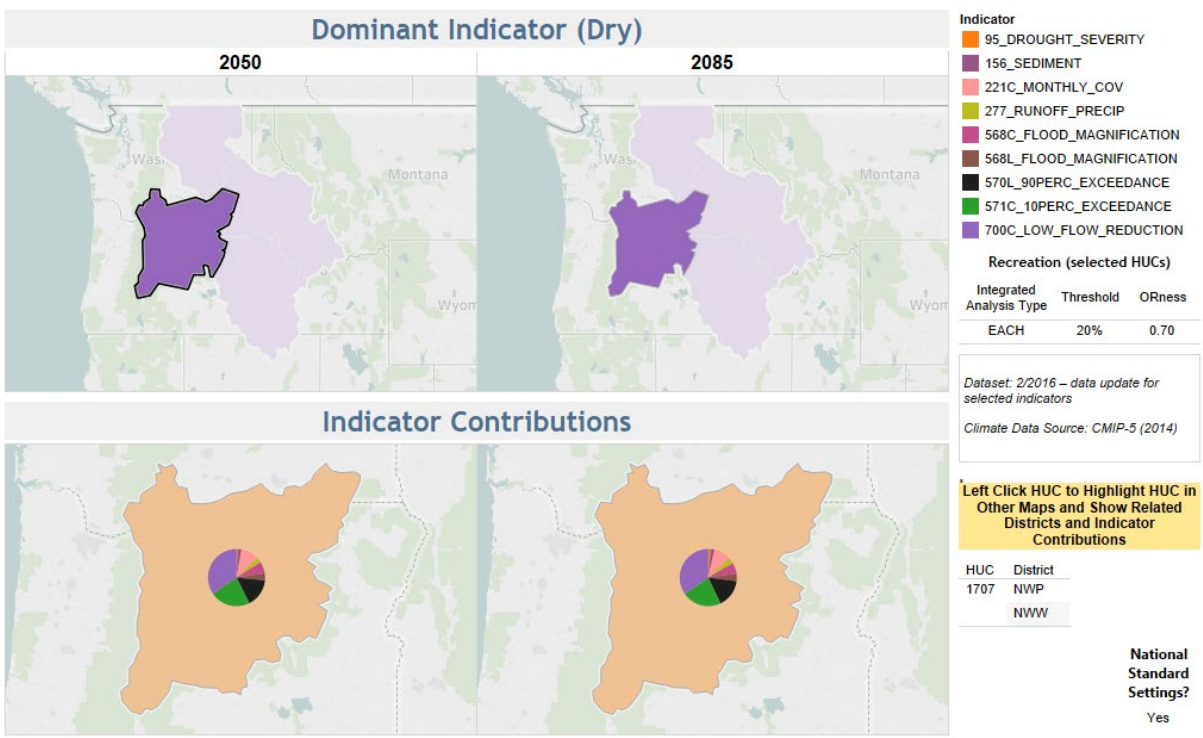

Figure 15 Dominant indicators for the Recreation business line are low flow reduction (35%), 10% exceedance (22.5%), and 90% exceedance (16%)

Another rendition of the vulnerability tool results are Figures 23, 24, and 25 summarizing each business line. Across the board for these business lines the WOWA score is < 70, indicating that HUC 1707 is not among the 20% most vulnerable for any relevant business line.

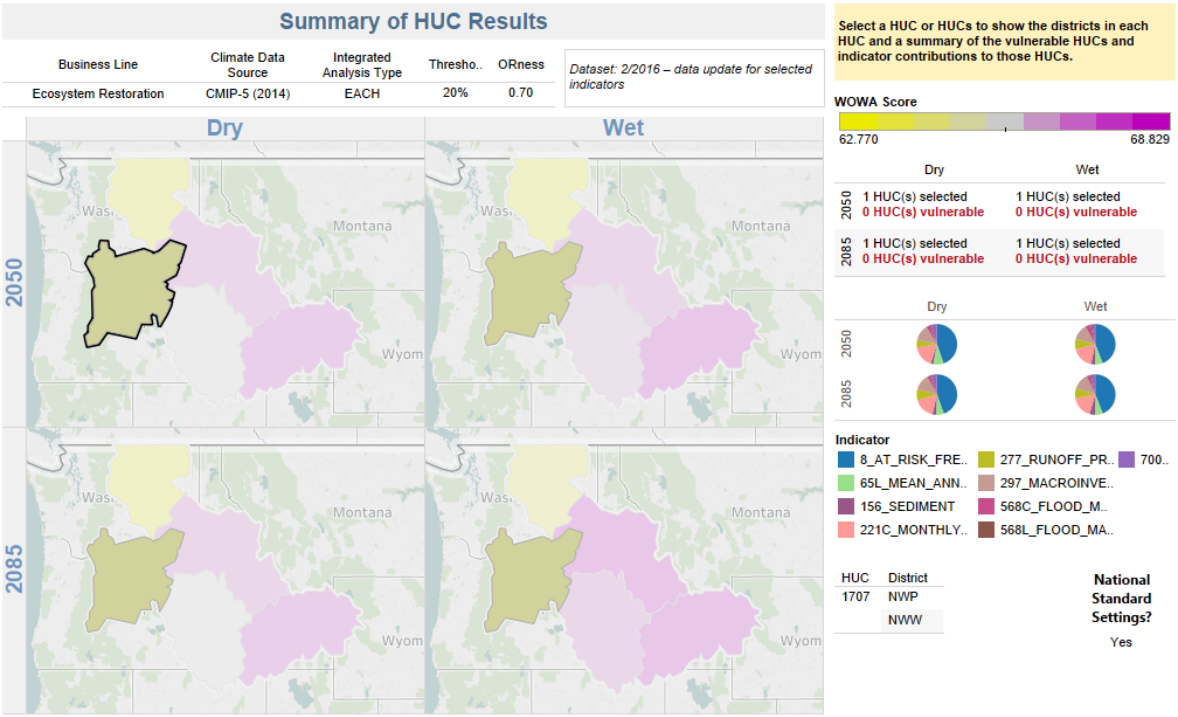

Figure 16 Vulnerability Tool assessment summary for Ecosystem Restoration

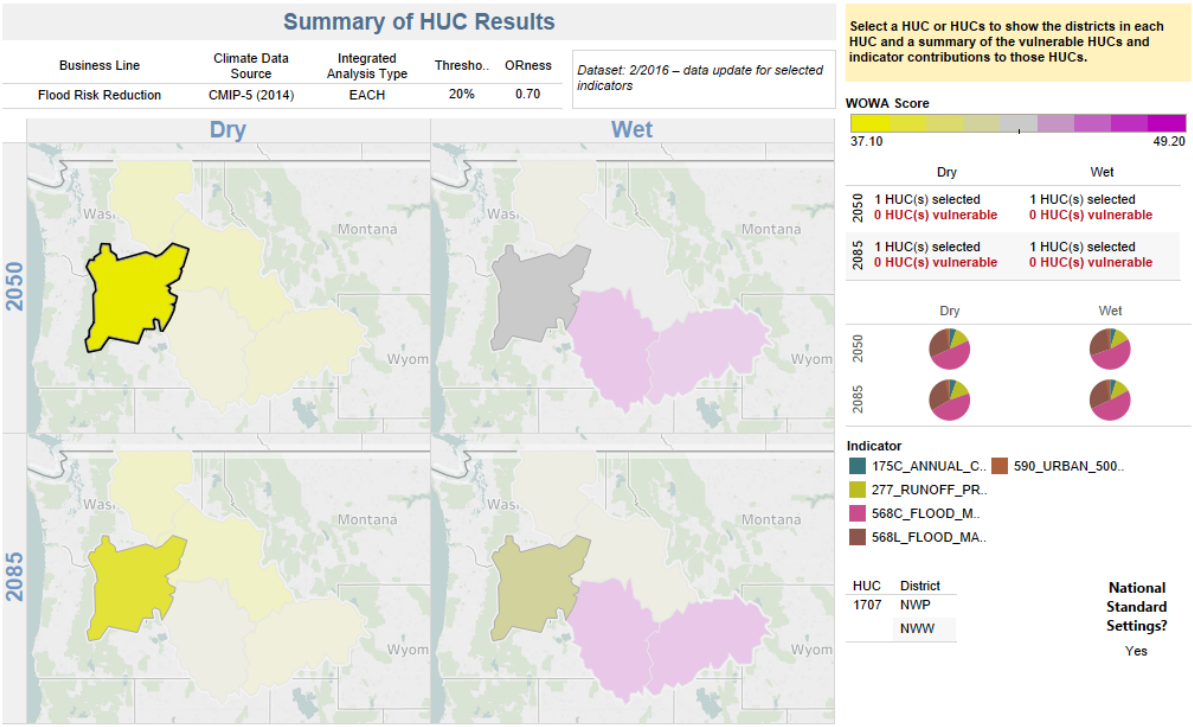

Figure 17 Vulnerability Tool assessment summary for Flood Risk Reduction

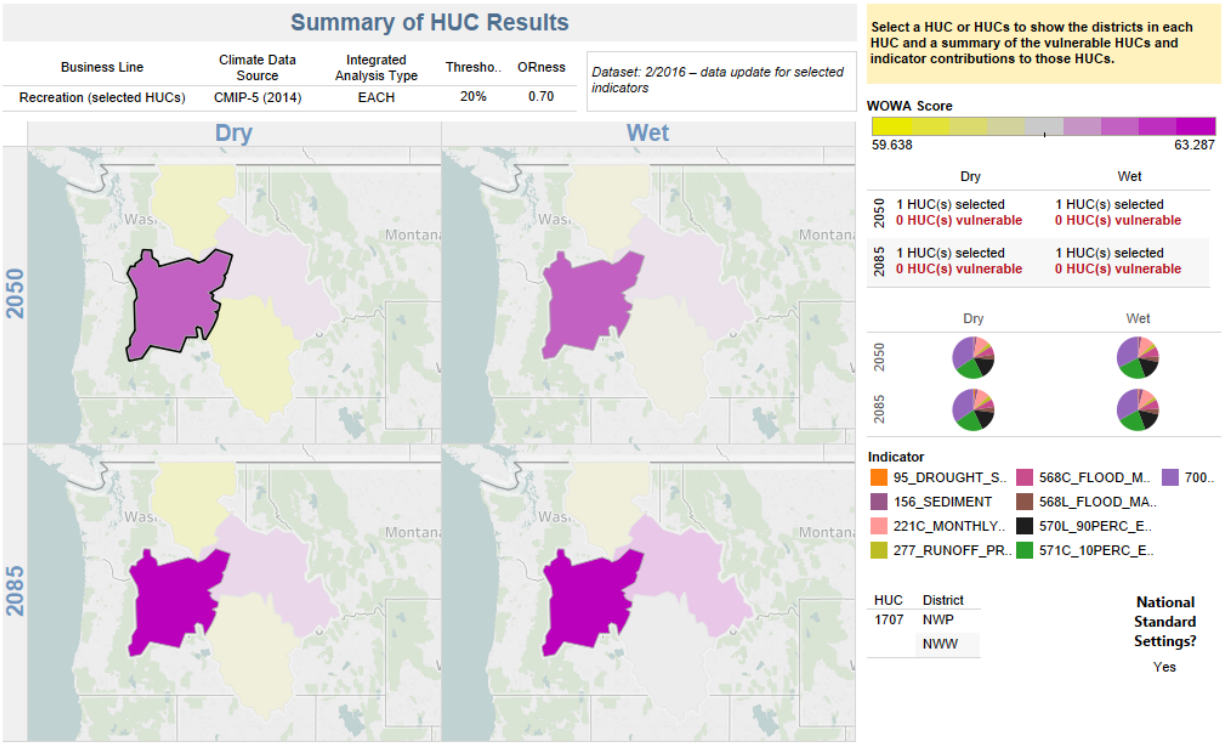

Figure 18 Vulnerability Tool assessment summary for Recreation

As apparent in the figures above and quantified in Table 4, the Mill Creek watershed, as part of HUC 1707 may be vulnerable to an increase in constraints in ecosystem restoration, flood risk reduction, and recreation due to climate change. There is only slight increase in vulnerability between 2050 and 2085 as well as a slight increase for the wet scenario. As indicated in Table 4. Projected Vulnerability with respect to HUC-4 Watershed and Business Line and Table 5 below, the HUC4 watershed encompassing Mill Creek (no. 1707 or the Middle Columbia) is most vulnerable for the ecosystem restoration and flood risk reduction. However, since the Mill Creek watershed (119 mi^2) is only 0.4% of the 29800 mi^2 HUC4 Middle Columbia subregion, and is tucked up adjacent to the Blue Mountains, the local climate change effects may be different than the larger scale sub-regional trends.

Table 4. Projected Vulnerability with respect to HUC-4 Watershed and Business Line

HUC-4 Watershed	Business Line	2050 Dry	2050 Wet	2085 Dry	2085 Wet
Middle Columbia (1707)	Ecosystem Restoration	64.94	64.95	64.88	65.25
	Flood Risk Reduction	38.30	42.61	38.96	42.16
	Recreation	62.24	62.36	63.17	63.29

Table 5. Comparison of Different Indicators for the Middle Columbia sub region watershed (HUC 1707)

		Year 2050		Year 2085		
Business Line	Indicator	Contribution to WOWA		Contribution to WOWA		Avg % Contribution
		Dry	Wet	Dry	Wet	
Ecosystem Restoration	Sediment	1.44	2.13	1.88	2.91	3
	Monthly Cover	11.22	11.37	10.87	11.35	*17*
	Runoff precipitation	4.07	5.46	5.85	5.47	8
	Macroinvertebrate	8.19	8.07	8.13	8.10	*12*
	Flood magnification-Cumulative	1.89	2.83	1.38	1.60	3
	Flood magnification - Local	0.85	0.89	0.91	0.94	1
	Mean annual runoff	5.30	3.97	4.05	3.99	7
	Low Flow Reduction	2.90	1.59	2.93	2.14	4
	At Risk Freshwater Plant	29.09	28.65	28.89	28.75	*44*
Flood Risk Reduction	Annual COV	2.0	2.12	2.05	2.08	5
	Runoff Precipitation	4.94	5.18	5.50	5.17	*13*
	Flood Magnification - Cumulative	19.23	22.48	18.33	21.41	*50*
	Flood Magnification - Local	11.04	11.81	11.89	12.37	*29*
	Urban 500-yr Floodplain Area	1.09	1.10	1.13	1.13	3

Recreation	Sediment	1.32	1.52	1.33	2.07	2
	Monthly COV	6.28	6.46	6.13	6.43	10
	Runoff Precipitation	1.89	1.98	2.10	1.52	3
	Flood Magnification - Cumulative	4.07	4.76	3.88	4.53	7
	Flood Magnification - Local	2.55	2.72	2.74	2.85	4
	90% Exceedance	10.04	9.96	10.09	9.96	*16*
	10% Exceedance	14.17	14.34	14.11	14.28	*23*
	Low Flow Reduction	21.50	20.36	21.89	20.99	*34*
	Drought Severity	0.45	0.28	0.90	0.65	1

Although the information is available online through USACE websites, this paragraph offers a brief description of terms in the above tables. For the ecosystem restoration business line, sediment is the ratio of the change in sediment load in the future to the present. Monthly cover is a measure of short-term variability in the region's hydrology based on runoff variance and mean. Macroinvertebrate reflects taxonomic diversity and composition. Flood magnification is the future flood runoff (monthly flow exceeded 10% of the time) divided by the flood runoff calculated from a base period. Cumulative includes upstream HUC4 watersheds whereas local restricts the calculation to this study's watershed. Low flow reduction is the change in low runoff, or monthly runoff exceeded 90% of the time. At risk plants is the percentage of wetland and riparian plant communities that are at risk of extinction, based on remaining number and condition, remaining acreage, threat severity, etc. For the flood risk reduction business line, the indicator annual COV is the long-term variability in annual runoff. Urban 500-yr floodplain area is the acres of urban area within the 500-yr floodplain. Under the recreation business line, the sediment indicator reflects the ratio of the future to present sediment load. Monthly COV is the monthly runoff variability. 90% and 10% exceedance are the occurrence of low flow and high flow, respectively.

From Table 5 wetland plants are most vulnerable in the ecosystem restoration business line, followed by monthly runoff variability and macroinvertebrates. Flood risk reduction dominant indicators are runoff precipitation and flood magnification. Finally, for the recreation business line the most vulnerability is seen in the change in low flow reduction followed by both 10% and 90% exceedance, reflecting the fact the flow will be more variable.

4. Future With-Project Condition

The alternatives chosen for the Mill Creek GI do not include a large reservoir or a large wetland. Currently the off-channel flood control reservoir Bennington Lake is approximately 8,300 ac-ft with 5 ft freeboard. No alternative considers creation of an impoundment significantly larger than this. Given this fact, one can surmise that the proposed project will have no direct significant effect on climate change during the lifetime of the project.

5. Description of Tentatively Selected Plan

Rather than the project having an effect on the climate, it is more likely that a changing climate will affect the project, or components of the project. Climate risks to the features and measures considered

as elements in the potential plans are listed in Table 6. This table depicts alternatives under consideration for the Mill Creek GI, their triggers, hazards, potential harm, and their qualitative likelihood.

Table 6 Potential climate risks applied to potential project alternatives

Feature or Measure	Trigger	Hazard	Harm	Qualitative Likelihood of Hazard Occurrence
Raised levee	Increased extreme flow event frequency, or longer duration precipitation events	Future flood volumes may be larger or longer-lasting than present Large flood volumes may occur more frequently	Flood waters may remain on the levee for longer durations, and more frequently, potentially damaging levee	Likely
Reservoir excavation between conservation pool and full pool	Increased storage exceeded by flood flow	More frequent large flood flows may sequentially fill extra storage space beyond capacity	Loss of property, shoreline damage, earthen dam instability	Unlikely, depending on amount excavated
Operational change- diversion trigger moved from 1400 to 2500 cfs	Increased peak runoff discharge and hydrograph timing change	Given more frequent extreme events, if diversion threshold increased, the main channel will experience bed-moving, channel shaping flows more regularly	Bank destabilization as channel dynamically responds to increased flow	Likely

6. Conclusions

Trends are noted for increasing water temperature, earlier hydrograph timing (decreased April 1 SWE), increased frequency of winter flows, decreased magnitude and longer duration summer flows. Various studies disagree about an increase in extreme precipitation with the majority leaning toward increased frequency (not magnitude) of extreme precipitation events, though variability is reported for precipitation generally. Specific magnitude and patterns of climate change impacts are not specified in this qualitative analysis. Through use of the online tools provided by USACE, no trend is apparent for the unregulated peak flows used as the basis for the Mill Creek GI. Based on the Middle Columbia HUC4 vulnerability analysis and literature review for the Northwest region, it can reasonably be projected that Mill Creek ecosystem will need to adapt or restructure in response to the gradual trend of warmer water temperatures, increased winter flow frequency, and less flow in the warm season. Additionally, the

project may need to be able to handle a higher frequency of extreme precipitation events. Each business line considered high priority for Mill Creek are considered below individually.

Flood risk reduction
Flood control is the primary reason for the Mill Creek project. From the tools mentioned in this study Mill Creek appears most vulnerable to increased flood frequency. If the trend of increased frequency of flood events continues project structures may experience more frequent loading.

Ecosystem restoration
Another business line relevant to the Mill Creek GI relates to water quality, stream fish habitat, and other ecosystem considerations. Project operations may need to adapt in consideration of salmonid habitat, flow magnitudes appropriate for fish passage, stream and shoreline habitat, and recreation opportunities. As far as Mill Creek GI scope allows, project alternatives may do well to incorporate elements that counteract the effect of increased temperature, lower flow or shifted hydrograph, and increased frequency of extreme events. Such measures could incorporate riparian vegetation for increased shading during typical flow magnitudes and perhaps even refuge in high flow events depending on how vegetation interacts with inundated hard structure. Further measures that could perhaps counteract an increasing temperature could include increased pool depth, or minimized shallow, slow flow exposed to direct sunlight. In light of these considerations, perhaps a more concentrated, faster and deeper channel would maintain lower channel temperature and improve valued species habitat.

Recreation
Lower base flows and increased flow variability may affect current patterns of recreation in and around the Mill Creek project. Timing of filling Bennington may need to be considered if the regional earlier runoff hydrograph timing pattern is expressed in the Mill Creek watershed.

In summary, future with-project conditions will likely have no foreseeable significant effect on climate change at the scale considered in this analysis when compared to the future without-project conditions. No new large reservoirs or wetlands will be created as part of the proposed project, and the project is on a different scale than what would impact the regional climate. However, the apparent and projected trends showing increasing temperature, earlier hydrograph timing, decreasing base flow magnitude, and increasing frequency of high flow events will probably effect the project. Timing may shift to an earlier peak, runoff from precipitation events may occur more frequently in winter, and in-channel ecosystem may need to adjust to a gradually warmer average temperature. Although the Northwest is relatively invulnerable to these changes compared to other U.S. regions as indicated by the above vulnerability analysis, resiliency to these effects of climate change in the Mill Creek watershed can be incorporated into the chosen Mill Creek GI project alternative. Alternatives connected with resiliency would be a benefit to the project whether or not conditions at the site shift over time due to climate change.

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