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 B
## ENGINEERING

This page intentionally left blank

DRAFT

**MILL CREEK FLOOD RISK MANAGEMENT GENERAL INVESTIGATION FEASIBILITY STUDY AND ENVIRONMENTAL ASSESSMENT REPORT – DRAFT**
**APPENDIX B, ENGINEERING**

## TABLE OF CONTENTS


**SECTION 1 - GENERAL ..... 1-1**
1.1 Format and Organization ..... 1-1
1.2 Project Overview ..... 1-1
**SECTION 2 - SURVEYING, MAPPING, AND OTHER GEOSPATIAL REQUIREMENTS ..... 2-1**
2.1 Levee Stationing ..... 2-1
2.2 Survey Data ..... 2-1
**SECTION 3 - GEOTECHNICAL ..... 3-1**
3.1 General ..... 3-1
3.2 Design Criteria ..... 3-1
3.3 Site Characterization ..... 3-1
3.3.1 Geologic Setting ..... 3-1
3.3.2 Seismic Site Classification ..... 3-2
3.3.3 Seismic Hazard Curve ..... 3-4
3.4 Completed Explorations and Testing ..... 3-5
3.5 Preliminary Design Parameters ..... 3-6
3.6 Improved Channel Features ..... 3-7
3.6.1 Mill Creek Levee Extension ..... 3-7
3.6.2 Channel Stabilizers ..... 3-7
3.6.3 Gabion Revetments ..... 3-7
3.6.4 Performance during the 1996 Flood ..... 3-9
3.6.5 Subsequent Maintenance and Repairs ..... 3-10
3.7 Existing Dams Reliability Analysis ..... 3-11
3.8 Existing Levees Reliability Analysis ..... 3-12
3.8.1 Levee Slope Stability Analysis ..... 3-12
3.8.2 Levee Seepage Analysis ..... 3-15
3.8.3 Levee Settlement Analysis ..... 3-16
3.8.4 Levee Seismic Analysis ..... 3-16
3.9 Proposed Measures ..... 3-16


DRAFT

3.9.1 Raise Existing Channel Levees ..... 3-16
3.10 Analyzed Measures Not in TSP ..... 3-18
3.10.1 Excavation of Bennington Lake Reservoir ..... 3-18
3.10.2 Setback Levees ..... 3-18
**SECTION 4 - ENVIRONMENTAL ENGINEERING ..... 4-1**
4.1 Use of Environmentally Renewable Materials ..... 4-1
4.2 Design of Positive Environmental Attributes into the Project ..... 4-1
4.3 Inclusion of Environmentally Beneficial Operations and Management for the Project ..... 4-1
4.4 Beneficial Uses of Spoil during Construction and Operation ..... 4-1
4.5 Energy Savings Features of the Design ..... 4-1
4.6 Consideration of Indirect Environmental Costs and Benefits ..... 4-1
4.7 Integration of Environmental Sensitivity into All Aspects of the Project ..... 4-1
4.8 Consideration of Environmental Problems on Similar Projects with Respect to the Environmental Review Guide for Operations (ERGO) ..... 4-2
**SECTION 5 - CIVIL DESIGN ..... 5-1**
5.1 Site Selection and Project Development ..... 5-1
5.2 Construction Access Ramps ..... 5-1
5.3 Real Estate ..... 5-2
5.4 Utilities and Relocations ..... 5-2
**SECTION 6 - STRUCTURAL ..... 6-1**
6.1 Concrete Channel Structural Reliability ..... 6-1
6.2 Criteria and assumptions ..... 6-7
6.2.1 Uncertainty Analysis ..... 6-7
6.2.2 Rehab Criteria ..... 6-8
6.3 Wall Analysis ..... 6-9
6.3.1 Fragility Curve Development ..... 6-10
6.4 Center Column Failure Analysis ..... 6-13
6.5 Ceiling Failure ..... 6-19
6.6 Concrete Channel Rehabilitation ..... 6-21
**SECTION 7 - ADDITIONAL REQUIREMENTS ..... 7-1**
7.1 Construction Procedures and Water Control Plan ..... 7-1
7.2 Flood Emergency Plans for Areas Downstream of Corps Dams ..... 7-1

DRAFT

7.3 Operations and Maintenance ..... 7-1
7.4 Access Roads ..... 7-1
7.5 Corrosion Mitigation ..... 7-1
7.6 Cost Estimates ..... 7-1
7.7 Schedule for Design and Construction ..... 7-1
7.8 Plates, Figures, and Drawings ..... 7-2
7.9 Data Management ..... 7-2
7.10 Use of Metric System Measurements ..... 7-2
**SECTION 8 - REFERENCES ..... 8-1**

## LIST OF TABLES

Table 2-1. Approximate Stationing of Pertinent Channel Features ..... 2-1
Table 3-1. Site Classification ..... 3-4
Table 3-2. Peak Horizontal Ground Acceleration Summary ..... 3-5
Table 3-3. Soil Properties Used in Analyses ..... 3-7
Table 6-1. Engineering Properties of Construction Materials ..... 6-8
Table 6-2. Left Super Elevation Depth from Top of Right Wall per DM7 Drawing for Varying Channel Flows ..... 6-17

## LIST OF FIGURES

Figure 1-1. General Features of the MCFCP ..... 1-1
Figure 3-1. Cross-Section of the Mill Creek Alluvial Fan through College Place; from the ..... 3-2
Figure 3-2. WUS Earthquakes with Mw ≥ 3.5 from 1850 to 2012 ..... 3-3
Figure 3-3. Seismic Hazard Curve for PGA, Site Class D ..... 3-5
Figure 3-4. Construction Photo Showing Trench Excavation for Revetment along Left Bank of Mill Creek, Looking Upstream ..... 3-6
Figure 3-5. Cross Section View of Wire Mesh (Gabion) Revetment along Levee Slope ..... 3-8
Figure 3-6. Additional Riprap Placed at Levee Toe from 1980s Rehab Work ..... 3-9
Figure 3-7. Gabion Baskets Placed on Top of Right Bank, Looking Upstream ..... 3-9
Figure 3-8. 2015 Repair Photos Provided by the MCFCZD Show Damaged Stabilizer Cap and Subsequent Repairs to Re-establish top of Stabilizer ..... 3-11
Figure 3-9. Left Bank Levee Slope Cleared of Stumps and Woody Vegetation ..... 3-11
Figure 3-10. Typical Levee Cross Section ..... 3-13
Figure 3-11. Approximate Location of Analyzed Levee Section (red line) ..... 3-14
Figure 3-12. Fragility Curve for Left Bank Levee Section at Station 292+25. ..... 3-15
Figure 3-13. Typical Section of Proposed Levee Raise ..... 3-17
Figure 6-1. Clinton St. Wall Failure December of 1946 ..... 6-3
Figure 6-2. Typical Wall Cross Section Drawing Showing Wall Underpinning ..... 6-4
Figure 6-3. Reinforced T-Wall Division Wall ..... 6-5

DRAFT

Figure 6-4. Reinforced T-Wall ..... 6-6
Figure 6-5. Wall Failure Sketch ..... 6-11
Figure 6-6. Wall Upstream of Otis Street ..... 6-12
Figure 6-7. Broken Drain behind the Wall Left Bank Downstream of Otis Street Bridge ..... 6-12
Figure 6-8. Wall Monitoring between Otis Street and Merriam Street ..... 6-13
Figure 6-9. Fragility Curve for a Channel Wall Failure Upstream of Otis Street Bridge ..... 6-13
Figure 6-10. Top Column Bearing Plate, No Welds or Bolts Connecting the Column to the Base Plate ..... 6-15
Figure 6-11. Pipe Column Center Wall ..... 6-16
Figure 6-12. Center Wall Column Wood Plank Connections ..... 6-17
Figure 6-13. Flow Profiles from DM7 ..... 6-18
Figure 6-14. Fragility Curve for the Center Wall Pipe Column Failure ..... 6-18
Figure 6-15. Channel Ceiling Condition Station 138+00 ..... 6-20
Figure 6-16. Fragility Curve for the Channel Ceiling Failure ..... 6-20

**ANNEXES**

Annex A, Reference Drawings
Annex B, Concept Drawings
Annex C, Levee Stability Analysis Results
Annex D, Structural Analysis Information

DRAFT

# ACRONYMS AND ABBREVIATIONS

ACE – Annual Chance Exceedance
AEP – Annual Exceedence Probability
CEUS – Central and Eastern United States
ECRA – Existing Condition Risk Assessment
EM – Engineer Manual
ER – Engineer Regulation
FOUO – For Official Use Only
FS – Factor of Safety
GIS – Geographic Information System
H&H – Hydraulics and Hydrology
LiDAR – Light Detection and Ranging
MCFCP – Mill Creek Flood Control Project
MCFCZD – Mill Creek Flood Control Zone District
MLV – Most likely value
Mw – Moment magnitude
NAD83 – North American Datum of 1983
NAVD88 – North American Vertical Datum of 1988
NEHRP – National Earthquake Hazards Reduction Program
NGVD29 – National Geodetic Vertical Datum of 1929
NID – National Inventory of Dams
NLD – National Levee Database
NSHMP – National Seismic Hazard Map Program
O&M – Operations and Maintenance
PGA – peak ground acceleration
PSHA – Probabilistic seismic hazard assessment
TOL – Top of levee
TSM – Taylor Series Method
TSP – Tentatively Selected Plan
USGS – United States Geological Survey
WUS – Western United States

DRAFT

This page intentionally left blank

DRAFT

# SECTION 1 - GENERAL

## 1.1 FORMAT AND ORGANIZATION

This document and the associated drawings and attachments comprise the Engineering Appendix to the Mill Creek General Investigation Feasibility Study and Integrated Environmental Assessment. This Draft Engineering Appendix presents and documents the feasibility level engineering and design to identify the Tentatively Selected Plan (TSP). This document has been formatted following Engineer Regulation (ER) 1110-2-1150 Appendix C - Content of Engineering Appendix to Feasibility Report. Several sections of ER 1110-2-1150 Appendix C are not applicable to this Study and are therefore not discussed.

## 1.2 PROJECT OVERVIEW

Major features of the Mill Creek Flood Control Project (MCFCP/Project) are a diversion dam and dike; Mill Creek channel improvements; the diversion intake structure and canal; an off-stream storage dam; a return channel; and an outlet canal. Portions of the MCFCP such as the diversion and storage dams are monitored under the district dam safety program. The National Inventory of Dams (NID) identification number is WA00348. Oversight of the improved channel portion downstream of the diversion dam is under the district levee safety program. The Left and Right Banks of the channel are listed under the National Levee Database (NLD) with system identification numbers of 6005000009 and 6005000010 for Left and Right Banks respectively. The levee sponsor, the Mill Creek Flood Control Zone District (MCFCZD), manages the operation and maintenance program for the non-federal portions of the channel, from downstream of the division works to the end of the project downstream of Gose Street Bridge. Figure 1-1 below shows the pertinent features of the MCFCP.

**Figure 1-1. General Features of the MCFCP**

# SECTION 2 - SURVEYING, MAPPING, AND OTHER GEOSPATIAL REQUIREMENTS

## 2.1 LEVEE STATIONING

The MCFCP channel is approximately 7 miles long, includes both the Left (South) and Right (North) banks of Mill Creek, and is one continuous system as it weaves west through the City of Walla Walla. The stabilized, or improved channel portion of the Project begins 500 feet downstream of the Gose Street Bridge and ends upstream at station 370+12, where the upstream end connects to the Diversion Dam. See Table 2-1 for approximate stationing for each feature and Annex A for maps showing stationing and approximate locations of pertinent channel features. Refer to main report Section 1.9.2.5 for additional details.

**Table 2-1. Approximate Stationing of Pertinent Channel Features**

| Feature | Downstream Station | Upstream Station | Approx. Length (miles) |
|---|---|---|---|
| Downstream Improved Channel | 6+00 | 108+00 | 1.9 |
| Concrete Lined Channel | 108+00 | 217+00 | 2.1 |
| Upstream Improved Channel | 217+00 | 369+70 | 2.8 |

## 2.2 SURVEY DATA

To support this study, 29 Geographic Information System (GIS) map products were generated covering a variety of alternatives and sub-alternative variations. Existing Light Detection and Ranging (LiDAR) data collected in 2010 and was used for elevations. This data is referenced to North American Datum of 1983 (NAD83) Washington State Plane South for horizontal coordinates and North American Vertical Datum of 1988 (NAVD88) for elevations. Typical levee, dam, and excavation cross sections were provided by geotechnical, structural, and hydraulic engineers to create three dimensional representations for the initial development of some of the structural measures (see main report Section 3.1).

All elevations shown in this Appendix are referenced to National Geodetic Vertical Datum of 1929 (NGVD29), unless otherwise stated as NAVD88. The elevation conversion used in this report is NAVD88 = NGVD29 + 3.3 feet.

DRAFT

This page intentionally left blank

DRAFT

# SECTION 3 - GEOTECHNICAL

## 3.1 GENERAL

This section presents geotechnical considerations and analyses on reliability of existing features based on historical data and recent investigations, as well as documenting assumptions for the geotechnical design of proposed features. The considerations consist of design for raising some reaches of existing channel levees and rehabilitation of several areas of the concrete channel to improve channel capacity and reliability.

## 3.2 DESIGN CRITERIA

Design of the selected alternative will follow current Corps criteria for geotechnical design of the applicable structure(s) as well as for any additional investigations and testing.

## 3.3 SITE CHARACTERIZATION

Site characterization data from previous studies is summarized below. This summary is not a lengthy discussion of regional geology or the long history of investigations. It is a focused summary to pull only the most significant information together to briefly describe the data used for evaluating the existing features and proposed structural measures.

### 3.3.1 Geologic Setting

The Project is located within the Columbia Basin Physiographic Province (also referred to as the Columbia Plateau), near the boundary with the Blue Mountains province to the east and southeast. The Columbia Basin extends to the north and west, and is bound to the north by the Okanogan Highlands and to the west by the Cascade Mountains. The broad, relatively flat Columbia Plateau is formed from a thick sequence of flood basalts (the Miocene age Columbia River Basalt Group), and is contained within a structural and topographic depression. Sediments unconformably overlie the Miocene basalt throughout much of the south and central Columbia Plateau and constitute a formation commonly referred to as the Ringold Formation. The Ringold Formation varies in composition from clays and silts to sands, gravels, and cobbles and can be unconsolidated to cemented. Thicknesses vary from several feet to hundreds of feet. Glaciations reached the northern edge of the Columbia Plateau during the Pleistocene Epoch. Outwash-derived sediments were deposited over the Ringold Formation and other fluvial and glaciofluvial deposits across the Columbia Plateau during this time. Remnant loess (windblown silt) of middle to late Pleistocene age found in the east and southeast of Washington State is referred to as the Palouse Loess. See Figure 3-1 below for generalized geologic cross-section.

DRAFT

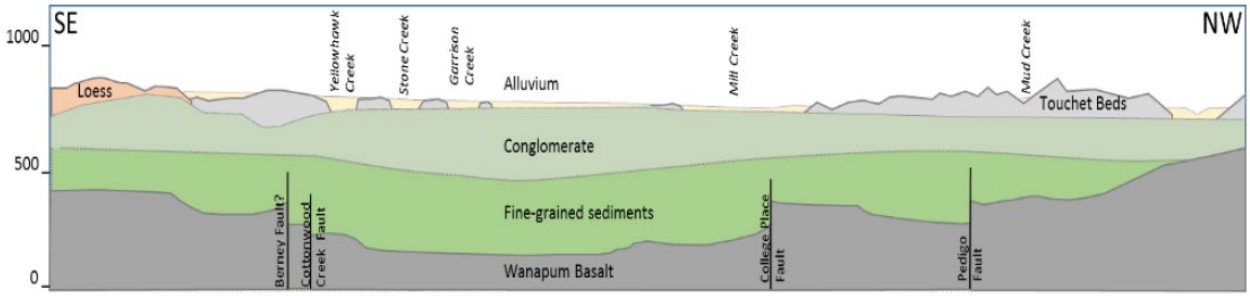


**Figure 3-1. Cross-Section of the Mill Creek Alluvial Fan through College Place; from the Southeast (SE) to Northwest (NW)**
Note: modified from Derkey et al. 2006

Local well records and project explorations near the stream channel indicate an inconsistent occurrence of the cemented materials. The older gravels occur about 14 to 20 ft below the more recent alluvium laid down by the present stream and overlie bedrock consisting of Columbia River Basalt. The most recent surface alluvium consists of a surficial layer of 2 to 10 ft of alluvial silt materials which overlies the more recent unconsolidated alluvial gravel and cobbles that form the streambed and banks and that were likely used in the construction of the diversion dike embankment, and the improved channel levees. Where silt has generally filled in the void space between the gravels and cobbles, the result is a less pervious material. The silt was not deposited in fast moving stream channels which have crisscrossed the river valley due to the braided nature of the stream channel. This results in some zones of high permeability, particularly in the recent alluvium, where the water table rises and falls with the level of the stream.

### 3.3.2 Seismic Site Classification

The 2014 update of the National Seismic Hazard Map Program (NSHMP) by the U.S. Geological Survey (USGS) (Petersen et al. 2014) provides consensus-based analyses of seismic hazard, using Central and Eastern United States (CEUS) seismic sources, Western United States (WUS) seismic sources, combined geologic/geodetic information, and revised ground-motion models. This report represents the most current assessment of seismic hazard for large regions of North America. The levee system is located at latitude 46.076577º, longitude -118.272872º (coordinates of the division works near Sta. 321+00) in the western United States.

The 2014 NSHMP seismic source model for the WUS considers both fault-based sources and seismicity-based background sources. Fault-based models are dependent on a slip rate that is based on a dated offset feature or on multiple measurements of geodetic strain rates across the region. The seismicity-based source model for the WUS assumes that future large earthquakes are more likely to nucleate near previous earthquakes with moment magnitude (Mw) greater than or equal to 3.5, as shown in Figure 3-2 .

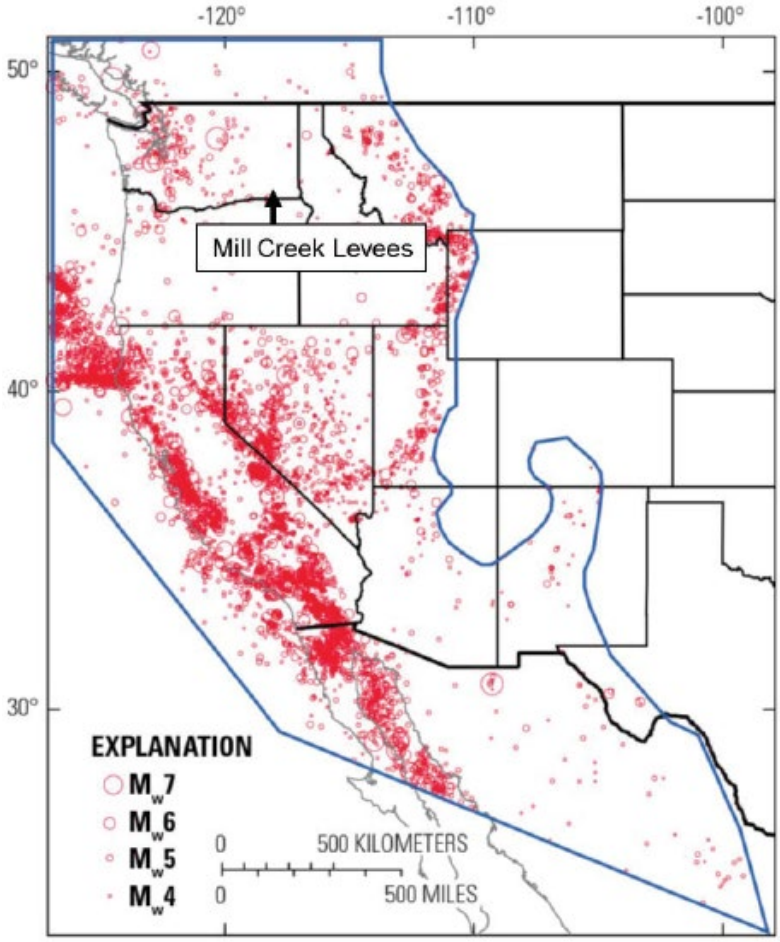


**Figure 3-2. WUS Earthquakes with Mw ≥ 3.5 from 1850 to 2012**
Source: Petersen et al. 2014

There were no seismic considerations in the original design, and no subsequent seismic evaluations have been performed. Although no seismic research has been performed for the Mill Creek Levees, research has been performed for other components of the MCFCP, such as the upstream storage dam and diversion dam. Seismic information acquired for the dams is relevant for the levees given the close proximity.

The Geological and Seismological Review, Lower Snake River Dams, McNary and Mill Creek, (Corps 1982), used deterministic methodology to develop a maximum credible earthquake (MCE) for the MCP. Results of the study indicate the Walla Walla Basin contains faults associated with the Wallula Fault System and the Hite Fault System. Geologically recent faulting near Buroker and historic seismicity at College Place and State Line indicate this area to be a credible seismic source in the plateau. Walla Walla Basin faulting is about 1.2 miles from the site of the MCFCP. The study estimated the MCE for the MCFCP to be a magnitude 6.2 event, based on epicenter distance of 1/2 the fault length of 9 miles. The estimated peak ground acceleration (PGA) from this event is 0.5g.

Previous assessments (Corps 2018) determined that the foundation soils were not likely prone to liquefaction due to the dense, heterogeneous mixture of cobbles, gravels, silts and some sands. Based on this review of available data, the levee's foundation materials were judged to correspond to Site Class D, according to the National Earthquake Hazards Reduction Program (NEHRP) shown in Table 3-1 below.

**Table 3-1. Site Classification**

| Site Class | Velocity ($v_s$) | Blow Counts (N or $N_{ch}$) | Shear Strength ($s_u$) |
|---|---|---|---|
| A. Hard rock | > 5,000 ft/s (> 1,500 m/s) | N/A | N/A |
| B. Rock | 2,500 to 5,000 ft/s (760 to 1,500 m/s) | N/A | N/A |
| C. Very dense soil and soft rock | 1,200 to 2,500 ft/s (360 to 760 m/s) | > 50 bpf | > 2,000 psf (> 100 kPa) |
| D. Stiff soil | 600 to 1,200 ft/s (180 to 360 m/s) | 15 to 50 bpf | 1,000 to 2,000 psf (50 to 100 kPa) |
| E. Soft clay soil | < 600 ft/s (< 180 m/s) | < 15 bpf | < 1,000 psf (< 50 kPa) |

Note: adapted from Table 20.3-1 of ASCE/SEI 7-16

### 3.3.3 Seismic Hazard Curve

A local (site-specific) probabilistic seismic hazard assessment (PSHA) has not been performed for this project. The mean seismic hazard curve for the peak ground acceleration (PGA) and Site Class D was generated using the regional (USGS 2014) PSHA, as shown in Figure 3-3. There is considerable uncertainty for an Annual Exceedence Probability (AEP) of less than 1/10,000. The extrapolation of the mean hazard curve to remote AEP (i.e., less than 1/10,000 AEP) is shown as a dashed line in this figure. The PGA corresponding to selected common values of return periods were interpolated from this mean hazard curve and are shown in Table 3-2.

DRAFT

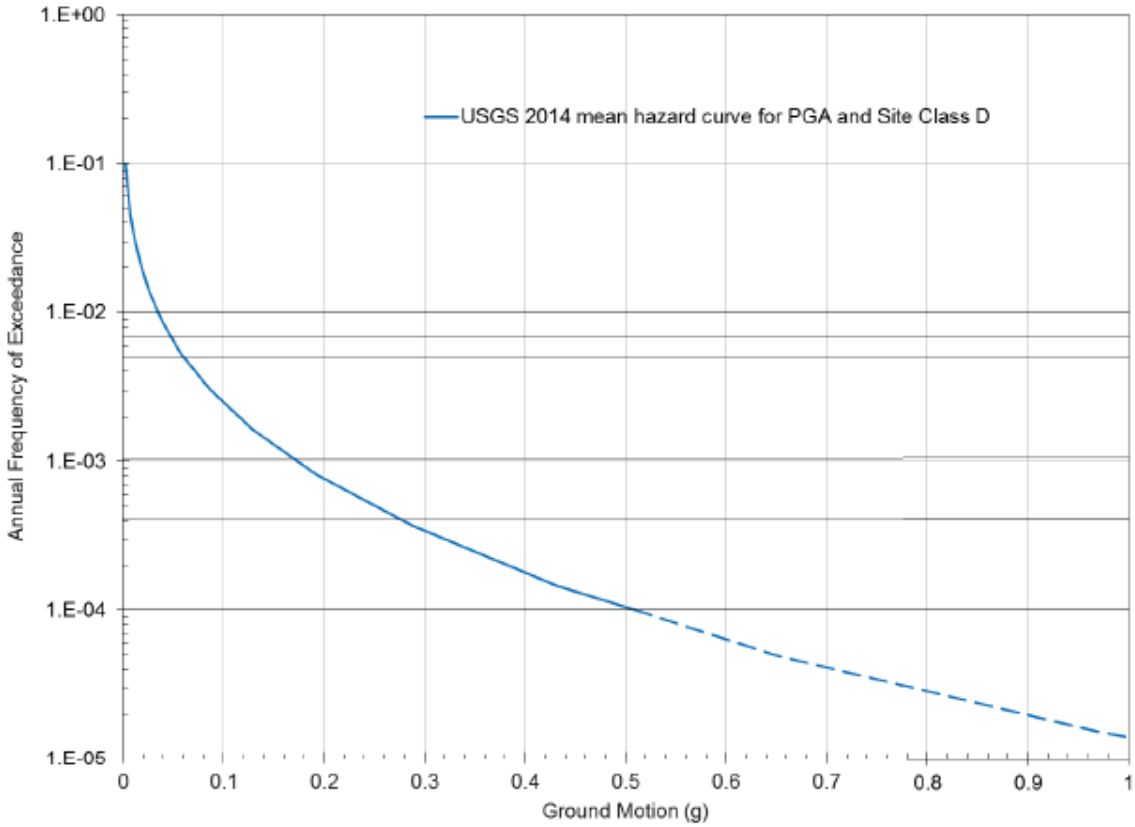


**Figure 3-3. Seismic Hazard Curve for PGA, Site Class D**
Source: from USGS 2014

**Table 3-2. Peak Horizontal Ground Acceleration Summary**

| Return Period (years) | PGA (g) |
|---|---|
| 100 | 0.03 |
| 200 | 0.06 |
| 500 | 0.11 |
| 2,475 | 0.28 |
| 10,000 | 0.51 |

Source: USGS 2014

## 3.4 COMPLETED EXPLORATIONS AND TESTING

Several geotechnical and geological investigations have been previously performed for the area and soils around the diversion dam and storage dam reservoir, as well as on the soils used in construction of the dam.

No new geotechnical explorations have been conducted for this study so far and very little information exists on levees downstream of the Division Works. However, it is assumed that foundation conditions are similar based on the regional geology and historic construction photos. Therefore, existing site data was utilized to develop reasonable estimates for this

study. Three shallow borings drilled on the federal levees in 2012 show poorly graded gravel with sand and cobbles (GP) in the levee foundation. Levee material at these locations show varying amounts of sandy silt (ML), organic silt (OL), sandy gravel (GP) and silty gravel with sand (GM).

Gradation tests of borrow materials for the levee embankments were not likely not performed during construction as the levees were constructed with materials from the adjacent channel. However, some composite samples were obtained from test pits dug in 2016 at a few locations on the federal levees and were sent for laboratory testing. Only a few tests were performed, such as gradation, Atterberg Limits, moisture content, and organic content. Based on this limited data the levees appear to contain a higher percentage of fine sand and silt. However, construction photos suggest there is a significant percentage of gravel and cobbles with some fines present in the levee foundation and river channel bottom (see Figure 3-4). No additional laboratory testing of samples has been performed for this study.

**Figure 3-4. Construction Photo Showing Trench Excavation for Revetment along Left Bank of Mill Creek, Looking Upstream**
Note: Date of photo 30 March 1936

## 3.5 PRELIMINARY DESIGN PARAMETERS

As part of the characterization for geotechnical analyses, estimated soil properties were compiled. The soil properties used in the various study analyses were derived from a range of typical values in published engineering resources as well as from existing testing data of explorations conducted in the general vicinity of the levees and diversion dam as described in Section 3.4 above. Soil properties used in this study are summarized in Table 3-3. These values can be refined for future design efforts if more site specific information is obtained.

For the levee stability analysis, the total unit weight was used for materials both above and below the phreatic surface, in accordance with Engineer Manual (EM) 1110-2-1902. Due to the

mostly granular composition of the soils and lack of strength testing data, cohesion was assumed to be negligible.

**Table 3-3. Soil Properties Used in Analyses**

| Soil Property[1] | Levee | Foundation | Wall Backfill |
|---|---|---|---|
| Total Unit Weight, γ (pcf) | 120 | 128 | 128 |
| Drained Friction Angle, Φ' (deg) | 30 | 34 | 34 |
| Cohesion, c (psf) | 0 | 0 | 0 |
| Estimated Horizontal Hydraulic Conductivity, $k_H$ (ft/sec) | $6.28 \times 10^{-4}$ | $1.09 \times 10^{-3}$ | $1.09 \times 10^{-3}$ |

## 3.6 IMPROVED CHANNEL FEATURES

The improved channel portions of Mill Creek consist of a trapezoidal section of varying bottom width having wirebound gravel-revetted levees and concrete capped stabilizers across the gravel channel bottom. Levee embankments have a top width of 6 to 13 feet and side slopes of 2H:1V. In more developed bank areas, the wirebound gravel banks are steepened to form a gabion sidewall having a riverside slope of 1H:2.8V. Existing channel improvements end at the Gose Street Bridge, and the unimproved natural channel extends about 5 miles further downstream to its confluence with the Walla Walla River.

### 3.6.1 Mill Creek Levee Extension

This levee extension is located on the right bank (or north side) of Mill Creek upstream of the diversion dike, which is an appurtenant structure to the diversion dam. Refer to Section 1.9.2.1 of the main report and Annex A of this appendix.

Levee screening was completed in 2014 and subsequent annual inspections confirm this levee is in good condition and is well maintained by the MCFCZD.

### 3.6.2 Channel Stabilizers

Refer to the H&H Appendix A, Section 6 for the assessment on the channel stabilizers.

### 3.6.3 Gabion Revetments

When originally constructed, slope protection on the river side of the levees was a system with a full-length tied-wire gabion slope mattress connected to a 2-foot wide, 3-foot tall gabion "fascine" at the toe (Figure 3-5). Cross-channel stabilizer structures, as described in Appendix A, Section 6, were constructed between the fascines at intervals on the order of 70 to 120 feet. In subsequent decades, steel corrosion and channel erosion (down cutting) caused gabion failures and rehabilitation efforts began in the early 1950s.

DRAFT

**Figure 3-5. Cross Section View of Wire Mesh (Gabion) Revetment along Levee Slope**

Service life of galvanized wire mesh is roughly 60 years. It is expected that areas of exposed wire bound revetment will likely continue to degrade and fail over the next 55 years. Based on minimum weight of revetment stone of 25 lbs, as specified in the 1940 solicitation for construction, instability would begin to occur at an average channel velocity of 10 feet per second if the wire mesh breaks or is compromised during a flood event. This was obtained from the stone stability chart in EM 1110-2-1601. However, channel rehabilitation completed in 1986 also added a full-length riprap toe revetment, providing corrosion resistant erosion protection along the zone of the levee where the water surface would cause accelerated corrosion of the wire mesh fascine and slope mattress (Figure 3-6). Class A riprap stone, which is 20 inches wide on average, was placed at the toe of the levee embankment, covering the fascine and rising two feet up the slope of the embankment. When observed during routine inspections, the MCFCZD also makes repairs to areas of broken wire mesh along the slopes and gabion walls.

DRAFT

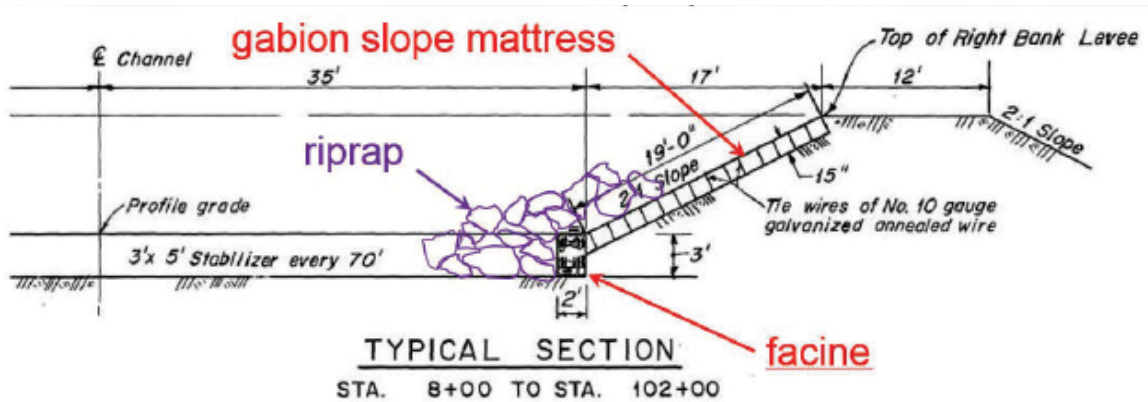


**Figure 3-6. Additional Riprap Placed at Levee Toe from 1980s Rehab Work**

Gabion structures were also constructed in 1980s along the riverside on the top the right bank downstream of Tausick Bridge (Figure 3-7). This structure was constructed to ensure freeboard during a flood event. Further downstream, between Wilbur Avenue and Roosevelt Street, the landside ground surface is higher than the existing gabion wall, therefore failure of a gabion wall section along here would likely lead to localized scour but no inundation.

**Figure 3-7. Gabion Baskets Placed on Top of Right Bank, Looking Upstream**

### 3.6.4 Performance during the 1996 Flood

Since completion of the full-length rehabilitation in the 1980s, Mill Creek has only experienced flows above 3,500-cfs once, which was the 1996 flood. While the observed performance was described as excellent, no scour survey or channel bed condition survey was conducted after

the 1996 flood. Scour damage to the riprap aprons was also not investigated. All of the stabilizers survived the flood event, apparently without significant displacement. However, there were more significant repairs needed at a few locations:

- The right bank levee riverside slope was eroded for several hundred feet immediately downstream of the diversion dam. This was repaired by construction of a full-height riprap revetment. It is not certain if the original construction wire bound slope mattress was damaged or removed. 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.
- Erosion occurred along the upstream end of the left bank concrete channel where the water surface was above the top of the concrete approach wall and caused erosion of the back yard area of a residence upstream of Roosevelt Street. This was repaired by construction of a mechanically stabilized earth (MSE) wall in 1998. It provided a concrete block face to the channel and construction involved filling the area behind the wall.

### 3.6.5 Subsequent Maintenance and Repairs

The MCFCZD and the Mill Creek Project staff have maintained the channel well over the years. Due to the primarily urban setting of this channel, there continues to be issues with woody vegetation, encroachments, and a lack of access for maintenance and flood fighting. These items are typically documented during the annual inspections and are addressed by the MCFCZD and Project staff as resources allow. Repairs of damaged stabilizer where occasional concrete failures expose the interior gabion mesh and cobbles are typically addressed quickly. The known repairs are limited to 5- to 15-ft lengths of stabilizer (see Figure 3-8). Areas of erosion downstream of the stabilizers are more difficult to evaluate due to water flow and depth, so a comprehensive condition survey has yet to be performed. Improved access for maintenance and flood fighting can be addressed under project rehabilitation of the concrete channel (see Section 5.2 below).

In 2015 and 2016, the Corps initiated a two-phased project on both the left and right banks of the federal levees to remove landside trees, regrade slopes using available on-site soils, and establish levee slopes that can be maintained more effectively (see Figure 3-9). The history of essentially forested levee slopes and the recently cut trees left a significant volume of woody matter in the soils and the earthwork included a significant grubbing effort to remove the root wads. Most of the final slopes were graded to 4H:1V to facilitate mowing, with a few areas graded at 3H:1V due to restrictions such as real estate boundaries or existing structures.

DRAFT

**Figure 3-8. 2015 Repair Photos Provided by the MCFCZD Show Damaged Stabilizer Cap and Subsequent Repairs to Re-establish top of Stabilizer**

**Figure 3-9. Left Bank Levee Slope Cleared of Stumps and Woody Vegetation**

## 3.7 EXISTING DAMS RELIABILITY ANALYSIS

Deficiencies of the diversion dam and dike, intake headworks and channel, and off-stream storage dam are evaluated by the district's dam safety program and repairs are made under federal operations and maintenance (O&M). These portions of the MCFCP are considered to be in good condition with only minor O&M recommendations. No further reliability assessments were made under this GI study as any future deficiencies that may arise on the dams will be addressed under the dam safety program.

In 2016, an Existing Condition Risk Assessment (ECRA) was completed to include both dams, as the operation of one impacts the other. Based on the results of the ECRA, the Mill Creek Project was recommended to be categorized as low incremental risk because the combination of life, economic, or environmental consequences with probability of failure is low to very low, and the dams may not meet all essential USACE guidelines.

## 3.8 EXISTING LEVEES RELIABILITY ANALYSIS

The Mill Creek levees were mostly constructed over 80 years ago in conjunction with the diversion dam and Bennington Lake storage reservoir. While there is information available on the pre-site conditions, construction methods and materials, various repair efforts and rehabilitations that have occurred in regards to the diversion dam and storage reservoir, limited information is available for the levees themselves, especially the non-federal portion of the levees. Some components of the flood risk reduction project existed before the authorized federal project was constructed. Therefore, as part of this study, a reliability analysis was performed on the levees to help inform future without project conditions. The publication *Factors of Safety and Reliability in Geotechnical Engineering* by J. Michael Duncan, 1999, was used as a general guide for this analysis.

A reliability analysis is a parametric study that quantifies the influence of varying levee material properties and geometry on the factor of safety (FS) for a particular failure mode (e.g. slope stability). Statistical methods are used to determine a probability of failure for a given set of levee material properties and geometries. Reliability analyses were conducted for a reference section to determine the probability of failure for slope stability failure modes using several flood loading conditions. Probability of failure (Pf) is one minus the reliability. The probability of failure versus water loading relationships for the representative levee section was then used to develop a fragility curve for the levee section analyzed. The upper bound, when probability of failure equals 1, is assumed to occur when the water level is 0.5 feet above top of levee (TOL), which is the threshold for failure due to overtopping. The lower bound is when probability of failure equals 0 (no load condition). This is consistent to what has been used in past risk assessments for levees.

### 3.8.1 Levee Slope Stability Analysis

The main portion of the levee reliability is to perform slope stability analyses to evaluate the landside slope stability of the levee embankment for various loading conditions. See Figure 3-10 for typical leveed channel section. Based on a review of levee construction history, past performance, subsurface conditions, and the results of previous studies, an analysis cross section was developed to characterize a “worst case” scenario for developing a fragility curve for use in determining economic damages due to breach. Levee crest, channel bottom, and water surface elevations for each load case were provided by H&H from current hydraulic modeling and LiDAR (Light Detection and Ranging) data. One section was developed for the left bank side of the non-federal levees upstream of the bridge at Tausick Way at approximate station 293+25 (see Figure 3-11). The top of levee (TOL) on the left bank is lower than the right bank by about 1.2 feet so the left bank was considered the controlling case at this location.

The stability of the landward levee slope was evaluated using the computer program SLOPE/W 2019. SLOPE/W is a two dimensional limit equilibrium stability analysis software program developed by GEO-SLOPE International, Ltd. (2012b). Spencer's Method was used for stability analysis and is a two-dimensional limit equilibrium method of analysis that satisfies all conditions of equilibrium (moment and force). Analyses of different loading conditions were performed on the representative cross-section shown in the SLOPE/W model (see Annex C for results). Annual chance exceedance (ACE) and water surface elevations for each flood flow (loading) was provided by H&H. The model uses the current geometry for a silt with sand and gravel levee prism atop a gravel with silt and sand foundation. Slip surfaces were specified to be at least 3 ft deep at one point along the slip surface as it is assumed that shallower slides would not cause failure and could be readily addressed by the project maintenance personnel. Estimated soil properties used for this analysis are shown in Table 3-3, above. Due to having predominantly non-cohesive, granular soils within the levee and foundation, rapid drawdown analyses were not performed.

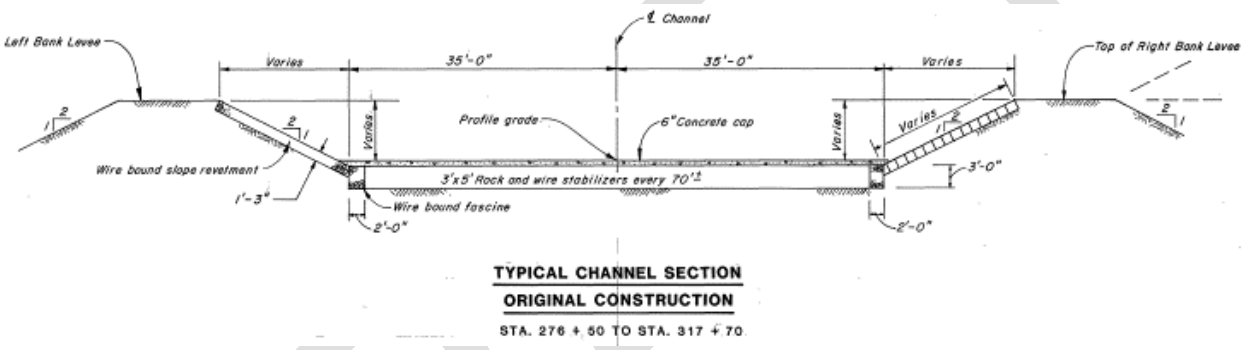


**Figure 3-10. Typical Levee Cross Section**

DRAFT

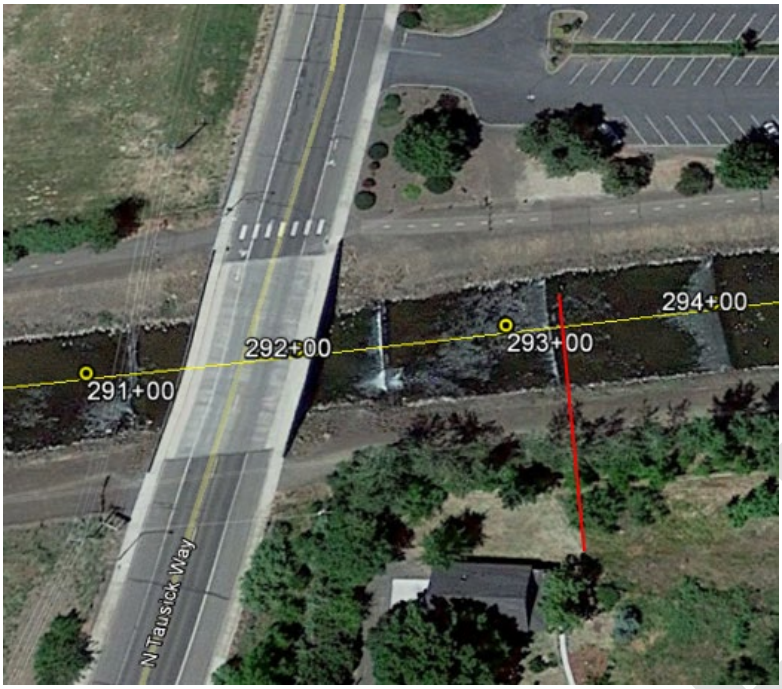


**Figure 3-11. Approximate Location of Analyzed Levee Section (red line)**

Gabion revetment mattress was neglected as it only exists on waterside slope and does not affect the stability results. Because the levees have occasional vehicle traffic for inspections and maintenance, a vehicle surcharge of 250 psf was applied on the levee crest for each applicable load case in the analysis.

Using the estimated soil properties (see Table 3-3, above) as the most likely value (MLV), estimated maximum and minimum values were used to determine the standard deviation for each soil property as described in Duncan, 1999. Levee geometries were varied by likely maximum and minimum values to account for uncertainty in the different levee reaches along the channel. Varied levee parameters include crest width and height. Summary tables of analysis parameters are shown in Annex C of this appendix. The analysis was then performed with each of the soil properties (unit weight and friction angle) at one standard deviation above the most likely value (MLV), at the MLV, and at one standard deviation below MLV while all other variables remained constant. Then in the analysis, soil properties were held constant at the MLV and the levee section was analyzed for the maximum and minimum for each levee geometry parameter (crest width and height). The resulting range of factors of safety for each variation was used to determine the reliability index for each load condition.

DRAFT

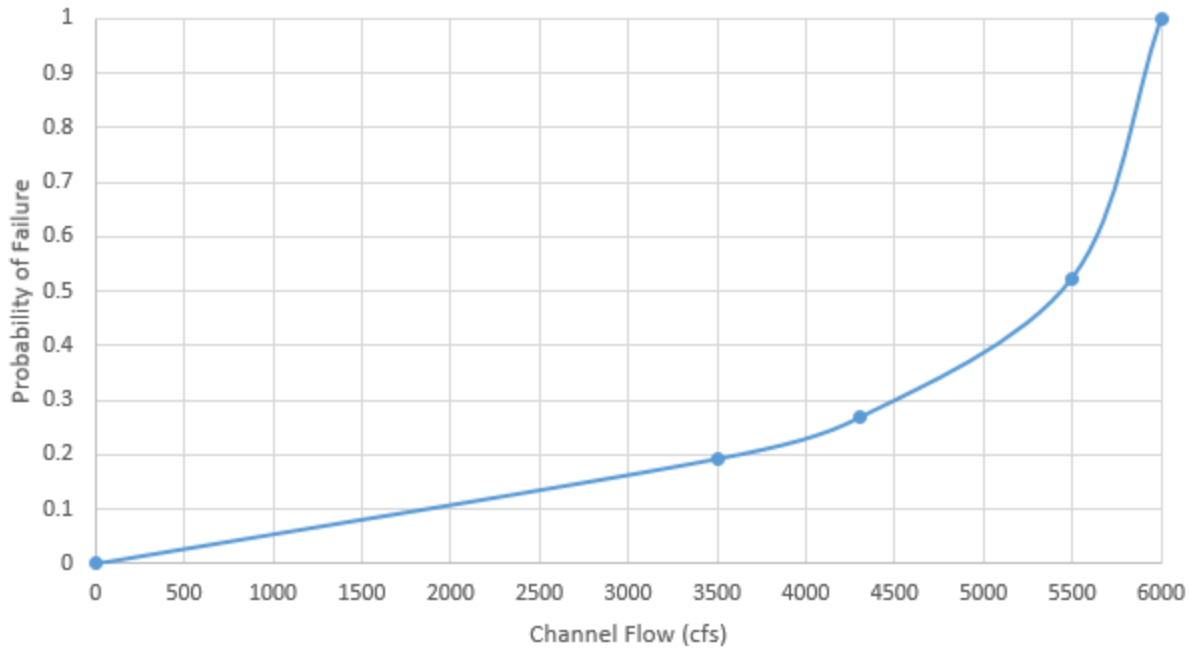


**Figure 3-12. Fragility Curve for Left Bank Levee Section at Station 292+25.**
Note: failure due to overtopping occurs at about 6,000 cfs at this location.

Annex C of this Appendix shows the results of the stability analysis for each load case. These results were used to develop the fragility curve for this levee section for existing conditions as well as for the proposed measures (Figure 3-12).

The non-federal levees are expected to perform well based on existing condition and past performance. The primary potential failure mode of the levees is overtopping. Therefore, based on available data, there are some reaches of levee that lack height to meet criteria to pass up to a 3,700 cfs 1-percent chance flood and be certifiable for the National Flood Insurance Program (see also H&H Appendix A, Section 3.3.2). The federal levees are expected to perform better due to having a wider crest as well as 3H:1V to 4H:1V landside slopes.

### 3.8.2 Levee Seepage Analysis

It is believed that dragline excavators and gasoline powered cable-shovels were used for levee construction and the levees were constructed with materials from the adjacent channel. It is also believed that excavation and placement would have routinely resulted in sufficient mixing to ensure relatively low permeability. Settlement, cracking, depressions, and repairs to address these issues are not documented in available project records. This is consistent with expectations that the fine-grained soils are well compacted and should have relatively low permeabilities. Therefore, detailed seepage analyses were not performed for the existing levees.

It should be noted that for the steady-state seepage condition to fully develop, the river stage would need to remain at the same level for a long period of time. In general, some of the levees may be vulnerable to internal erosion once steady state conditions develop, but

permeabilities are low, and flood durations are not long. Furthermore, the height of the levees is generally not substantial enough to generate sufficiently high differential heads through the embankment to allow for initiation and progression of concentrated leak erosion or other internal erosion mechanisms.

### 3.8.3 Levee Settlement Analysis

There is no history of significant static settlement over the current life span of the MCFCP levees. Based on previous annual inspections, there has been no indication of distress related to settlement. The levee system embankment and foundation soils consist of silts, gravels, and sandy gravels, and the expected rate of settlement for these soil types would be rapid and would have occurred mostly during or immediately after construction. Minimal continued settlement is anticipated along these levees given they are more than 80 years old.

### 3.8.4 Levee Seismic Analysis

Due to the relatively low heights, wide cross-section, and low liquefaction potential, the levees were not analyzed for seismic loading. Foundation materials consist of a dense, heterogeneous mixture of cobbles, gravels, silts, and some sands that are not likely prone to liquefaction. Although there is some uncertainty due to lack of foundation data, if a seismic event was large enough to cause deformation to the levees, they could be repaired quickly if inspected right after the event occurred. There is also a very high unlikelihood of a large seismic event (ACE 1/10,000) occurring during a large flood event (ACE 1/200).

## 3.9 PROPOSED MEASURES

The current TSP includes a combination of structural measures such as a levee raise of up to 1.5 feet, and some areas of concrete channel rehabilitation. For the proposed measures, geotechnical considerations are discussed below. Refer to the main report, Section 4, for overall details of the TSP.

### 3.9.1 Raise Existing Channel Levees

The current TSP estimates the some reaches of levee will need to be raised by a maximum of 1.5 feet in order to meet the 3,700 cfs 1-percent chance flood criteria for the National Flood Insurance Program. This will also allow for an increase in flow conveyance capacity through the leveed areas, which improves operational flexibility. Figure 3-13 shows a typical section for levee raise. Drawings in Annex B show areas where raising levees may be required. Levee heights were estimated from previous LiDAR survey information, with the “worst case” height being used at each reach for the analysis (see H&H Appendix A, Section 3.3.2). As models are updated and refined during this study, the locations and required heights for the levee raise will change.

Very little excavation, other than clearing and grubbing is anticipated for raising the existing channel levees. For levees with smaller required raises, additional road gravel or asphalt

concrete pavement will be used to meet criteria. Some additional riprap may be necessary above the existing rip rap toe to address displaced or missing stone. Currently it is assumed that the existing wire bound slope revetment and gabion walls will not need to be extended or modified under this proposal plan.

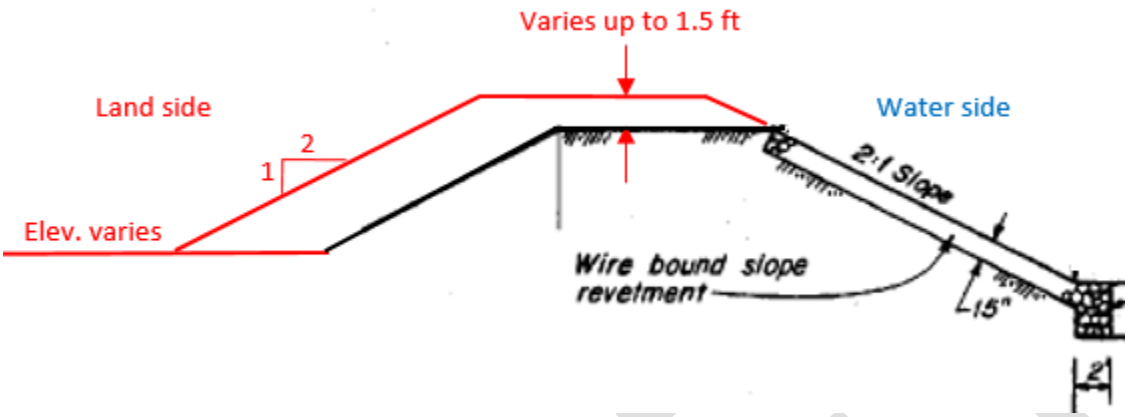


**Figure 3-13. Typical Section of Proposed Levee Raise**

A 1V on 2H slope is generally accepted as the steepest slope that can easily be constructed and ensure stability of any riprap layers (EM 1110-2-1913). To provide access for normal maintenance operations and flood fighting operations, minimum crown widths of 10 to 12 ft are commonly used. These widths are about the minimum feasible for construction using modern heavy earthmoving equipment and should always be used for safety concerns (EM 1110-2-1913). Stability analysis of a levee section, located at station 100+10 (left bank), with the new width and height was performed to check that a minimum stability FS of 1.3 can be met with existing 2H:1V landside slopes and an average height increase of 0.7 feet (see Figures C-29 and C-30 in Annex C).

The available as-built drawings show that the existing levees meet these recommendations for 1V on 2H side slopes, but will be verified during design. Field verification will also be needed for 10 ft crown width but it is likely the most of the existing levees already have this due to some past widening efforts. If there are areas that do not meet the minimum 10 ft-width, they will be widened during the levee raise. However, consideration will be given to allow a minimum 8-ft crest width where clearances or improvements will not permit the wider section, if this situation is encountered.

There are potential onsite borrow sources within the Project boundaries near Bennington Lake, but gradation and other index testing will need to be performed as a minimum to confirm the material is suitable for the levee raise. Local commercial sources can also be used to supply the material needed for the levee raise if necessary.

## 3.10 ANALYZED MEASURES NOT IN TSP

### 3.10.1 Excavation of Bennington Lake Reservoir

Additional excavation of the existing storage dam reservoir (Bennington Lake) was determined to be a feasible measure to allow for an increase in storage for flood operations above conservation pool at elevation 1,205 ft but was excluded from the current TSP due to significant costs for material disposal (see main report Section 3.1.8).

Excavation in Bennington Lake would be expected to produce mainly fine-grained silt materials, sandy gravels, and cobbles. There is potential for re-use of this excavated material for use in raising the levees or for wall backfill. Further investigation of excavated material would be necessary to confirm the material is satisfactory for use as levee embankment fill or wall backfill. Due to past agricultural history of the area, it is assumed that the excavated material is not contaminated, so special handling and off-site disposal would not be required.

No specific disposal areas have been identified at this time as this measure is not included in the current TSP. It is assumed that the disposal areas would mostly be located off-site due to the lack of available real estate within the Project boundaries. Therefore, a higher disposal cost is anticipated due to the need to haul the majority of the material several miles to off-site disposal areas. However, the local sponsor could also elect to take some suitable material for use in other projects. Other options could include the local landfill, which could use some material for capping, as well as nearby private landowners. It is also assumed that existing routes are available and are adequately upgraded and maintained. Economic feasibility of truck hauling is also dependent on the following considerations:

- Haul distance to all disposal areas.
- State highway and safety regulations, which cover a variety of elements (such as gross weights of trucks and weight per axle).
- Emission and noise standards.
- Local ordinances designating truck routes.
- Weight limits on bridges.

### 3.10.2 Setback Levees

Refer to main report Section 3.1.3.1 for discussion on the setback levees. It is assumed that if this measure is added later that the new setback levees would consist of similar material and cross section as the existing levees.

DRAFT

# SECTION 4 - ENVIRONMENTAL ENGINEERING

Environmental objectives and requirements described herein will be fulfilled by adopting and enforcing prudent and reasonable measures to avoid impacts and by the completion of measures described in the main report for this study. Only applicable sections are described below.

## 4.1 USE OF ENVIRONMENTALLY RENEWABLE MATERIALS

There is little opportunity to incorporate renewable materials in this project. The majority of the work will consist of raising certain reaches of existing channel levees and rehabbing portions of the concrete channel. One of the major construction materials will be concrete, while not considered to be renewable, it could be composed of recycled concrete.

## 4.2 DESIGN OF POSITIVE ENVIRONMENTAL ATTRIBUTES INTO THE PROJECT

The levee land side slopes will be mostly vegetated utilizing a grass and wildflower seed mix. For the concrete channel portion, feasible modifications to the low flow channel will be considered during the final design to improve fish passage.

## 4.3 INCLUSION OF ENVIRONMENTALLY BENEFICIAL OPERATIONS AND MANAGEMENT FOR THE PROJECT

Refer to main report Section 5.

## 4.4 BENEFICIAL USES OF SPOIL DURING CONSTRUCTION AND OPERATION

It is anticipated that very little spoil material will be generated under the proposed plan, therefore, no significant beneficial use of spoil was evaluated. Although minor quantities are anticipated, some of the spoil material can be reused as fill material on other projects within and around the city and county. The plan for disposal of spoil material will avoid and minimize adverse environmental impact to the maximum extent practicable.

## 4.5 ENERGY SAVINGS FEATURES OF THE DESIGN

Due to the scope and nature of this flood risk management project, there are no feasibly obtainable energy saving features available.

## 4.6 CONSIDERATION OF INDIRECT ENVIRONMENTAL COSTS AND BENEFITS

Refer to main report Section 5.

## 4.7 INTEGRATION OF ENVIRONMENTAL SENSITIVITY INTO ALL ASPECTS OF THE PROJECT

Environmental sensitivity will be incorporated into the design and construction of the project to the maximum extent practicable.

DRAFT

## 4.8 CONSIDERATION OF ENVIRONMENTAL PROBLEMS ON SIMILAR PROJECTS WITH RESPECT TO THE ENVIRONMENTAL REVIEW GUIDE FOR OPERATIONS (ERGO)

Any issues or concerns noted in the Environmental Review Guide for Operations (ERGO) will be addressed through the Environmental Assessment in the main report.

Construction activities for concrete channel rehabilitation, and improved channel levee raising will cause minor vegetation loss and temporarily increased water turbidity. Turbidity and impacts to aquatic habitat will be minimized by timing construction during summer low flows.

DRAFT

# SECTION 5 - CIVIL DESIGN

## 5.1 SITE SELECTION AND PROJECT DEVELOPMENT

In order to find a solution for flood risk management, various channel alignments and new sites were evaluated to determine the available measures (see main report Section 3). The current TSP does not require new alignments of the channel or any relocation of facilities.

## 5.2 CONSTRUCTION ACCESS RAMPS

During construction the contractor will be required to construct access as needed to perform rehabilitation work on the concrete channel. Some of the access ramps can be made permanent so that the MCFCZD can use them for improved access into the channel for future maintenance and emergency operations.

The MCFCZP and USACE identified the following locations for potential permanent access points below. See also Plate B-11 in Annex B for additional details.

*Upstream of concrete channel*

- Francis Ave near station 221+50: construct a concrete ramp with gate where the existing gravel ramp is located; provide a crossing in the near vicinity unless access is provided on both the north and south sides of the channel.
- Construct a concrete ramp on the south side of the channel between Wilbur and Roosevelt, off of Hobson Street (in lieu of crossing, above).

*Along concrete channel*

- Upstream of Division Street along the right bank at Wildwood Park between stations 195+00 and 201+00. The wall is low and a section could be cut out for constructing a ramp and bulkhead into the channel.
- Fourth Avenue Bridge near station 128+00: If this bridge is removed and not replaced, this may provide a good location for access; construct a concrete ramp and bulkhead and provide a crossing in the near vicinity unless access is provided on both the north and south sides of the channel.

*Downstream of concrete channel*

- Washington Park near station 107+00: similar to the Francis Ave location, construct a concrete ramp with gate along end of concrete wall; provide a crossing in the near vicinity unless access is provided on both the north and south sides of the channel.

DRAFT

Other potential locations for consideration are listed below:

- On the left bank just upstream of Park Street, near station 165+00. There is a 6 foot high channel wall which could be cut out and a ramp built down into the channel. This location was recently used for temporary access for a Tri-State Steelheaders fish passage project.
- End of Blue Street on the right bank downstream of Roosevelt St near station 209+00.
- End of Marcus Street near First Congregational Church at station 160+25: construct a concrete ramp and bulkhead and provide a crossing in the near vicinity unless access is provided on both the north and south sides of the channel. Due to limited clearance along a residential street, this location will likely not be considered due to potential high construction and real estate cost.

## 5.3 REAL ESTATE

This project will require the acquisition of real estate in order to raise some of the levees as well as for rehabilitating the concrete channel and constructing access ramps. Because some levees are being raised and extended landward, up to 5 feet landward of the existing toe of levee may need to be acquired to incorporate the raised levee footprint.

## 5.4 UTILITIES AND RELOCATIONS

For the TSP, existing sanitary sewer, potable water, gas, electric and telephone lines will have to be located to determine if they will be impacted. Utilities located in the vicinity of the project will be identified by using GIS files provided by Walla Walla County. In general, if utilities have to be relocated, quantities will reflect an in-kind replacement, meaning that the same size and type of material would be utilized in the relocation of a utility to accommodate the proposed channel rehabilitation work.

DRAFT

# SECTION 6 - STRUCTURAL

## 6.1 CONCRETE CHANNEL STRUCTURAL RELIABILITY

This study of the Mill Creek concrete channel walls is to determine the reliability of the channel walls, mainly for the duration of the study life assumed to be 50 years. The second part of the study is to provide rehab recommendations to keep the walls performing satisfactorily for the next 50 years. The majority of the concrete channel is at least 80 years old. The age of those portions of the channel flood walls constructed using a combination of stone, mortar, and concrete, as well as building foundations, are unknown but are likely significantly older. Per discussions with building owners, some of the masonry walls were likely constructed as early as 1899.

There are very few design and construction details available for the channel as it was constructed by multiple entities over a significant period of time. Based on a walk through inspection of the channel, the walls of the channel are generally in satisfactory or good condition. The channel has been reliable even during significant events such as the 1996 floods. There are some areas with cracking and deterioration that should be addressed, and these have been highlighted in the recommended rehab areas (Annex B, Plates B-1 thru B-4). There are also some locations where drain piping through the walls has been damaged and settled, leaving the walls susceptible to becoming surcharged from drainage (see Figure 6-7). MCFCZD has been doing regular maintenance to keep the channel in operating condition.

Many localized repairs have been completed at various locations along the channel some as recent as 2012 near 6th Avenue. As the channel ages, the level of required maintenance will increase. There are some areas where the walls appeared to have rotated into the channel, in some cases as much as several inches. These areas that have noted rotations do not appear to have experienced any notable recent movements. One location along the left bank between Otis and Merriam Streets is currently being monitored for wall movements (see Figure 6-8 for current survey information). The study has focused on the channel walls and not as much on the channel covering as the top of the covered portions of the channel are outside of the main flood protection channel. However, some of the covered areas in poor condition have been considered a possible initiator to a failure of the system. A failure of one of these areas during a significant flood event could lead to debris blocking the channel and subsequent out-of-bank flooding and inundation.

The concrete channel has been very reliable with few known failures occurring in the channel during its history. The main one was in December 1946, which resulted in wall failure at the Clinton Street Bridge (Figure 6-1), due mainly to scour and undermining. Corps of Engineers memorandum documentation dated December 17, 1946, stated that several channel wall sections at other locations were also damaged from the event. Following the failure, a significant amount of underpinning was completed in the concrete channel to help prevent a reoccurrence of the event. Figure 6-2 shows an example of underpinning that was completed. In addition to the underpinning, a concrete channel floor was added in 1948 which further

helped stabilize the existing channel walls. The limited number of failures is likely due to the fact that there have been relatively few high flow flood events and the continued maintenance that has been performed over the years.

Figures 6-2 through 6-5 are examples of channel wall cross section drawings that have been found looking through County records. These cross sections, along with drawings from the 1948 channel rehab and DM7, were the main source of information for the analysis of the channel and determining the worst case or governing conditions. The bridges across the channel (such as the one shown in Figure 6-1) are monitored and maintained by the City or the County and they were not determined to be a likely cause for failures in the channel leading to flooding.

Based on inspections of the building basements, the masonry walls are typically very thick, approximately 2 ft thick at the top with assumed thicknesses of 3 ft or more at the base. The masonry walls in many locations are the foundation walls for the downtown buildings. These masonry walls are only located in the covered portion of the channel. Typically, they are in good condition; the mortar is sound and hard, and appears to have good bond strength to the stones. No signs of settlement or movements to these walls were noted during the inspections. Based on discussions with building owners in the areas where these walls form basements, there has been no report of leakage through these walls into the adjacent basements. These walls are also braced at the top with channel bridges or building floors. The building bridges above the masonry walls provide a significant vertical force on the walls that provide a stabilizing force for the walls. Based on these findings and the inspections, the masonry walls are not assumed to be a failure location for the channel.

DRAFT

**Figure 6-1. Clinton St. Wall Failure December of 1946**
Note: Approximately 300 ft of Wall Was replaced.

DRAFT

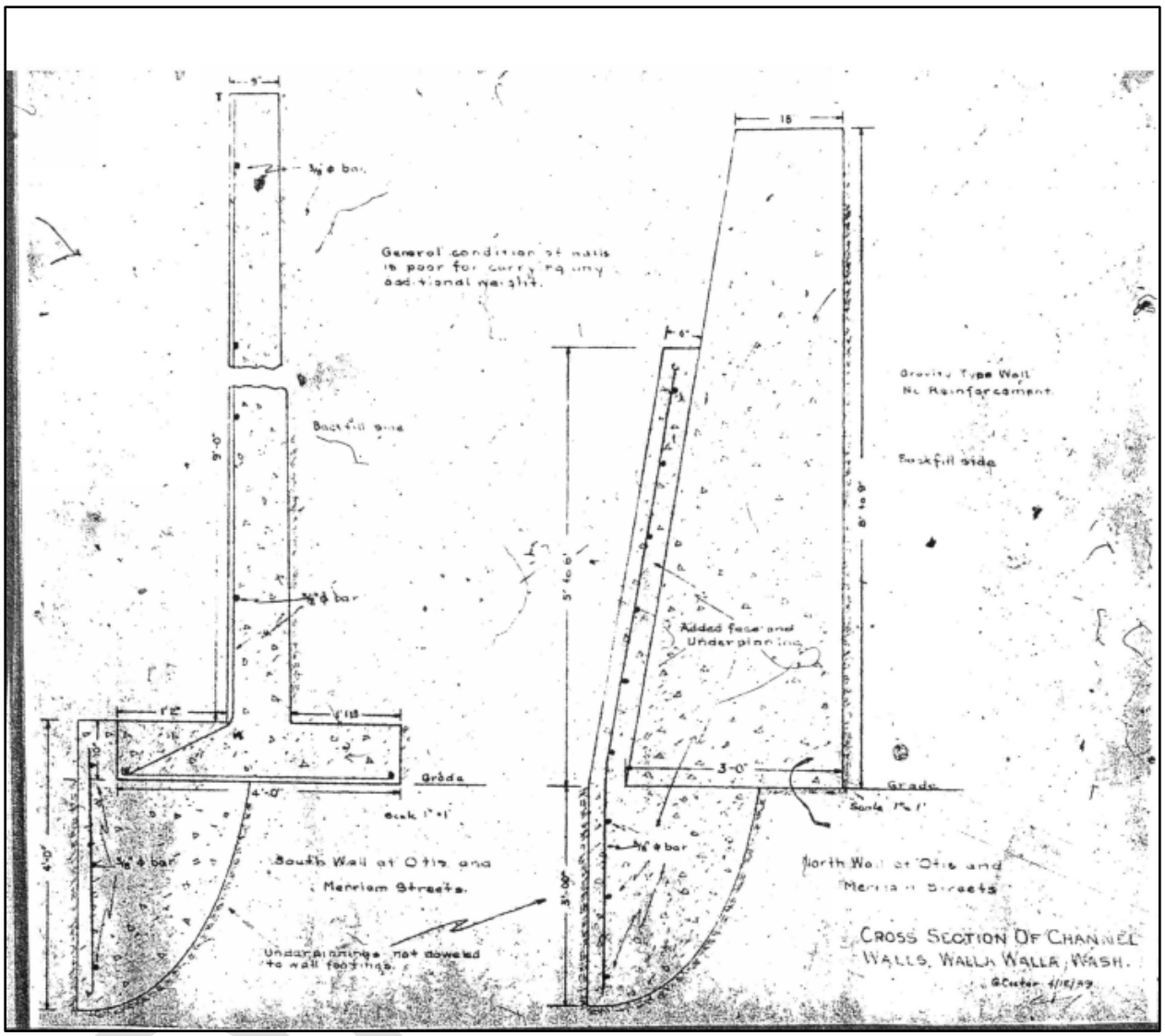


**Figure 6-2. Typical Wall Cross Section Drawing Showing Wall Underpinning**

Note: This drawing indicates two wall types Type 1: Lightly reinforced T-wall cross section shown on the left. The note to the right of the T-wall indicates the "general condition is poor for carrying additional weight." Type 2: Mass concrete or gravity wall typical cross section shown on the right.

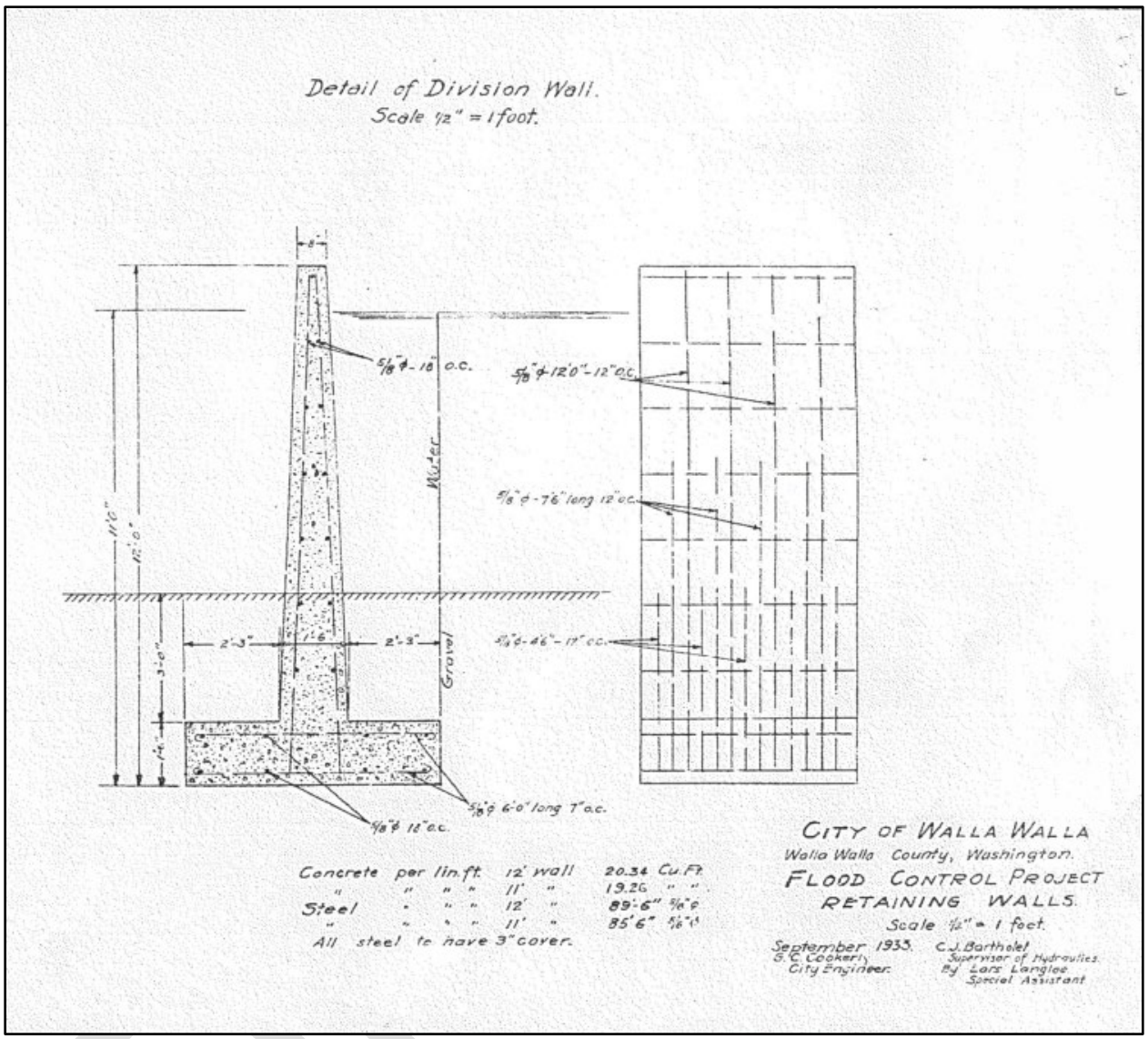


**Figure 6-3. Reinforced T-Wall Division Wall**

Note: the location where this cross section is applicable is not shown; it is assumed to be typical of several locations.

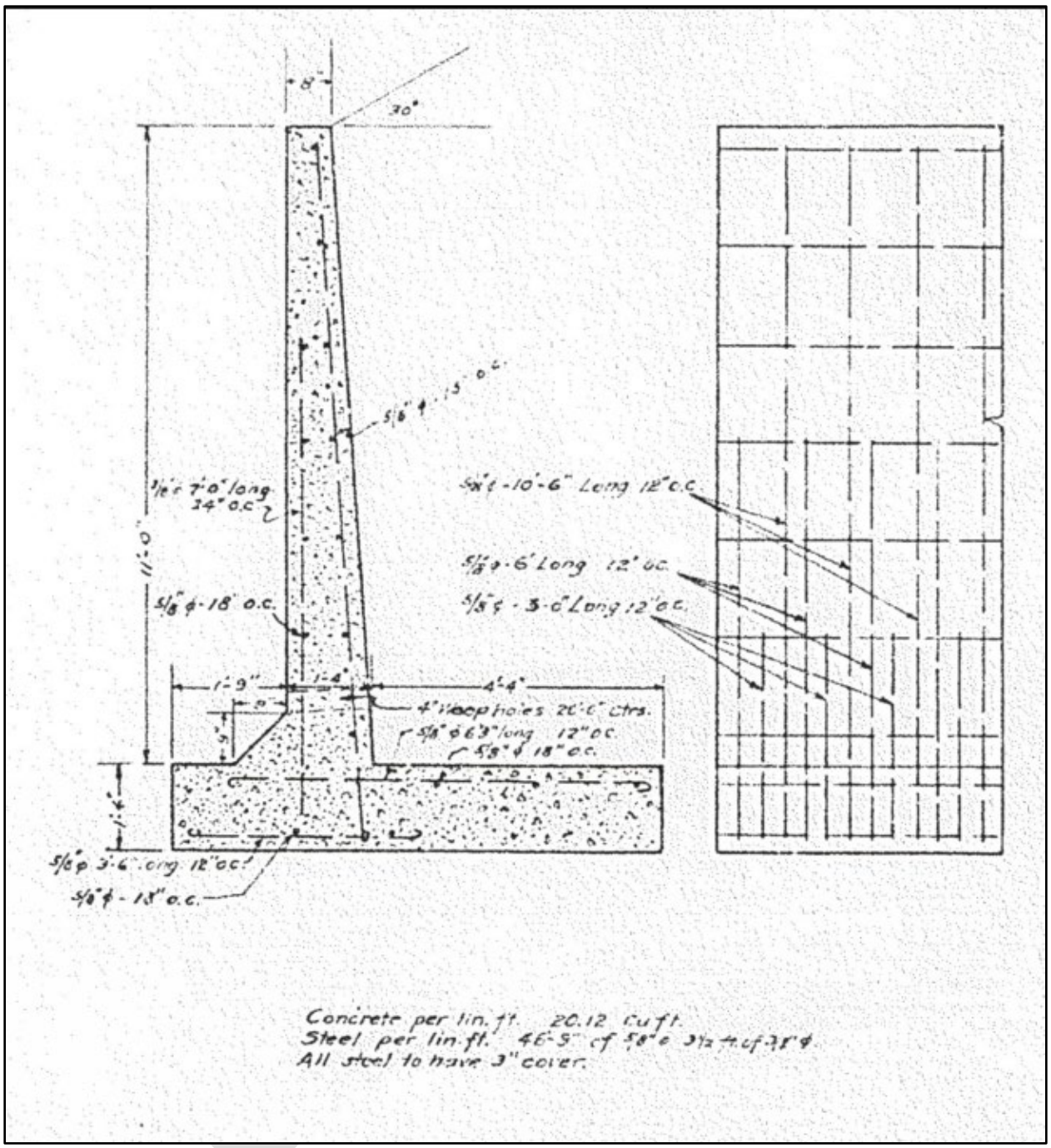


**Figure 6-4. Reinforced T-Wall**

Note: the location where this cross section is applicable is not shown; it is assumed to be typical of several locations.

## 6.2 CRITERIA AND ASSUMPTIONS

### 6.2.1 Uncertainty Analysis

For structures not meeting deterministic strength and stability criterion, a risk and uncertainty analysis was performed. A Taylor Series Method (TSM) of analysis was used in the calculation of structural risk and uncertainty. The TSM is appropriate when data is normally distributed, when parameters display a linear relationship, and when degradation over time is not a consideration. Because of the limited availability of data and with no information to suggest otherwise, an assumption of normal distributions for input data is reasonable and consistent with guidance provided in ETL 1110-2-547 (paragraph B-6.c). No safety factors were used in the analysis of the walls.

#### *6.2.1.1 Risk Calculations Existing Structures*

- For strength calculations, uncertainty is measured by applying a mean and standard deviation to the concrete and steel strengths. The selected mean and normal standard deviation are based on engineering judgment and information published in Reliability Based Design in Civil Engineering by Milton E. Harr.
- For stability calculations, uncertainty is considered by applying a mean and standard deviation to the soil unit weight and shear strength, and is based on values provided by the geotechnical engineers working on the study.
- Failure is defined as the capacity to demand ratio (factor of safety) less than 1.0, or in other words, when the demand (loads) exceed the capacity (structural or geotechnical).

#### *6.2.1.2 Material Properties*

Based on age of the concrete and the fact that no design data was available some typical strengths of concrete for that time frame were assumed. The American Concrete Institute recommended the use of a 3,000 psi concrete strength around the 1940's and 1950's, the typical timeframe of construction for most of the levee structures in these feasibility studies. For earlier concrete strengths, little information exists. It is currently assumed that 2000 psi concrete strengths are appropriate. If additional research information is discovered, this value will be updated. Based on FEMA 310, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength. For reinforced concrete structures, Harr suggests a 14% standard deviation (Harr 1987). Based on dated photos the walls were existing in the 1930's some drawings that were found for bridges dating back to 1920s show the walls to be existing. For the this study it was assumed that the strength of the concrete would be in line with the 2,000 psi min based on the 1900's-1920's.

- Concrete Strength Variation
- 1940's-1950's: $-\sigma = 3225$, $\mu = 3750$, $+\sigma = 4275$ (3000 psi min)
- 1900's-1920's: $-\sigma = 2150$, $\mu = 2500$, $+\sigma = 2850$ (2000 psi min)

### 6.2.2 Rehab Criteria

All design work for rehabilitation of the channel will be based on the below referenced edition of the codes, or most current editions of the following codes.

ACI 318-14, Building Code Requirements for Structural Concrete

ACI 350-06, Code Requirements for Environmental Engineering Structures and Commentary

ACI 350.4R-04, Design Considerations for Environmental Engineering Concrete Structures

ASCE 7-16, Minimum Design Loads for Buildings and Other Structures

AWS D1.1, 2015 Structural Steel Welding Code

AWS D1.6, 2017 Structural Welding Code Stainless Steel

EM 1110-2-2104, Strength Design for Reinforced Concrete Structures

EM1110-2-2502, Retaining and Flood Walls

IBC 2018 International Building Code

**Table 6-1. Engineering Properties of Construction Materials**

| **Properties** | |
|---|---|
| All Cast-in-Place Structures | |
| New concrete touching or containing water | f'c=5,000 psi |
| Modulus of elasticity (E) | 3,600,000 psi |
| Poisson's ratio | 0.2 |
| Steel Reinforcement: | |
| ASTM A615 Grade 60 | fy=60,000 psi |
| Structural Steel and Connectors: | |
| ASTM A36 (carbon steel: angles, plates, bars) | fy=36,000 psi |
| ASTM A500 Grade B (carbon steel: round hss) | fy=42,000 psi |
| ASTM A992 (carbon steel: w sections) | fy=50,000 psi |
| ASTM A53 Grade B (carbon steel: pipe) | fy=35,000 psi |
| ASTM A108 (carbon steel: shear stud connectors) | fu=65,000 psi |
| ASTM A325 (carbon steel: high strength structural bolts) | fu=120,000 psi |
| ASTM A240 (stainless steel) | fy=30,000 psi |
| ASTM A276 (stainless steel) | fy =30,000 psi to 45,000 psi depending on type selected |

ASTM = American Society for Testing Materials.

f'c = Specified compressive strength of concrete.
fy = Specified yield strength.
fu = Specified ultimate strength.

#### *6.2.2.1 Geotechnical Data*

Data for foundations and soil properties are detailed in section 3.5.

#### *6.2.2.2 Seismic design*

Seismic design will be based on site classifications and accelerations from Site Characterization Section 3.3 of this appendix

## 6.3 WALL ANALYSIS

Two different approaches were used to analyze the channel walls for reliability, wall stability and wall structural failure. Outside of the covered portion of the channel, there are two types of cantilever walls, T-walls and mass gravity walls. T-walls must be stabilized by the footing below grade, and mass gravity walls by a mass of concrete. The first step in the analysis was to look at stability of the walls for different loading events. In general, when the channel is full of water during a flood event, the water pressures will be pushing outward on the walls and loading them into the soil behind them. Water loads in the channel would typically not lead to any instability of the walls or failure due to shear or bending. The hydraulic loading of the walls from inside the channel is not a controlling load. There are a few locations where the walls border building basements; these areas are mainly near the downtown covered portion of the channel. In these few areas the water loads from a high flow event directly apply bending and shear loads to the walls. However in these areas the walls are stabilized at the top by the channel cover. There are also vertical stabilizing forces from the weight of the walls and floors above.

The typical controlling load for the cantilever channel walls is the result of a surcharge behind the walls from a saturated soil condition. This surcharge would be the result of a high flow event in the channel that could surcharge the soil by leaking through cracks or penetrations through the walls. Assuming the water level in the channel could drop relatively fast, the saturated soil would surcharge the wall leading to a wall failure. For this analysis, it was assumed that the surcharge water elevation behind the wall was the same as the high water elevation from the flood event.

The stability of several wall sections was analyzed using CTWAL, a CASE (Computer Aided Structural Engineering) program produced by the U.S. Army Engineer Research and Development Center to analyze the stability of T-Walls. The analysis is based on criteria from EM 1110-2-2502. Based on these stability checks, the walls were not meeting current stability requirements even for normal load cases with no additional surcharges from either water or additional live loads. These stability checks were based on assumptions for the footing base widths. Since the stability analysis is highly sensitive to parameters such as the width of the

base, and there is very little information to verify the dimensions, it makes the stability analysis susceptible to a significant error.

Another modeling approach beyond the standard T-wall or gravity wall stability calculations was to model the walls with the floor acting as a horizontal support at the base of the cantilever wall see Annex B, Plate B-9 for a graphic of the structural model. This was the model that was chosen to look at the wall failures. Since the walls are continuously braced by the floor, the failure modes were assumed to be failure in bending or shear at the base of the wall next to the top of the floor (See Figure 6.5).

The wall analysis was performed with different water elevations in the channel, resulting from flood events with flows of 3,000 cfs to 5,400 cfs. These values were used since the water profiles were available for those flows from DM7. Because the failure of a channel wall is based on a surcharge from a drawdown of a flood event, it would require a multiple peak flood event for failure to occur. The first high flow peak could be the failure initiator, followed by a second peak which would lead to flooding out of the channel with a blockage.

The seismic loading of the walls was not evaluated for this study. This study is focused on flood loadings. The likelihood of a seismic event occurring within same time as a short duration flood event is extremely low and can be ignored.

### 6.3.1 Fragility Curve Development

Because the capacity of the concrete channel is about 5,400 cfs and the channel is below grade, it was determined by the hydraulics design branch that after a wall failure, a blockage of 50 percent would be required to get a significant amount of flow out of the channel in an event around 3,000 cfs or more. It was assumed that a typical wall failure could result in a portion of the wall failing that could be 50 – 100 ft in length. After a failure of a portion of the wall, a second peak of water flowing through the channel would be partially blocked by the failed portion of the wall in the channel. The likelihood of a wall failure progressing to a flood event is not something that can be calculated. To determine this probability of wall failure to progress to a 50 percent blockage, the process of expert elicitation was used. The value for this progression to occur was originally assumed to be 0.01 by the expert elicitation team. Based on further discussions with the team and some additional team members, the original failure progression probability was revised from 0.01 to 0.20. This reflects more discussion about the likelihood of addition debris from trees near the possible failure location, providing a higher likelihood of a blockage at the Otis Street Bridge. The clearance under the bridge could also increase the likelihood of a blockage.

Summary of assumptions for the wall analysis:

- Loading-Water elevation profiles that were used were based on DM7 and later adjusted per the most recent hydraulic modeling for the specific worst case areas.

- Thickness of the walls was estimated based on the available cross section drawings. In areas that appeared to be mass concrete the assumed wall thickness at the point of failure was assumed to be 18 to 36 inches.
- Width of the base footings was not used for the final analysis since stability was not assumed to control with the walls being braced by the channel floor.
- Strength of the concrete was assumed to be 2,500 psi with a range of standard deviation of 350 psi.
- Reinforcement in the walls is minimal to none in some locations. A check of the minimally reinforced section indicated that the strength of the reinforcement in bending, was less than the strength of the plain concrete. Strength calculations for wall bending and shear were based on plain concrete strengths using ACI-318 2005
- Height and the depths of the footings and channel floor, including underpinning, were based on the 1948 drawings.
- Active soil pressures where used for the analysis of the walls. Typically for concrete walls at rest pressures would have been used based on the stiffness of the wall. Based on the movements that appear to have occurred in the past, likely in part to the excavation below the walls during underpinning, and inadequate base widths for the footings, active pressures appear to be more likely.

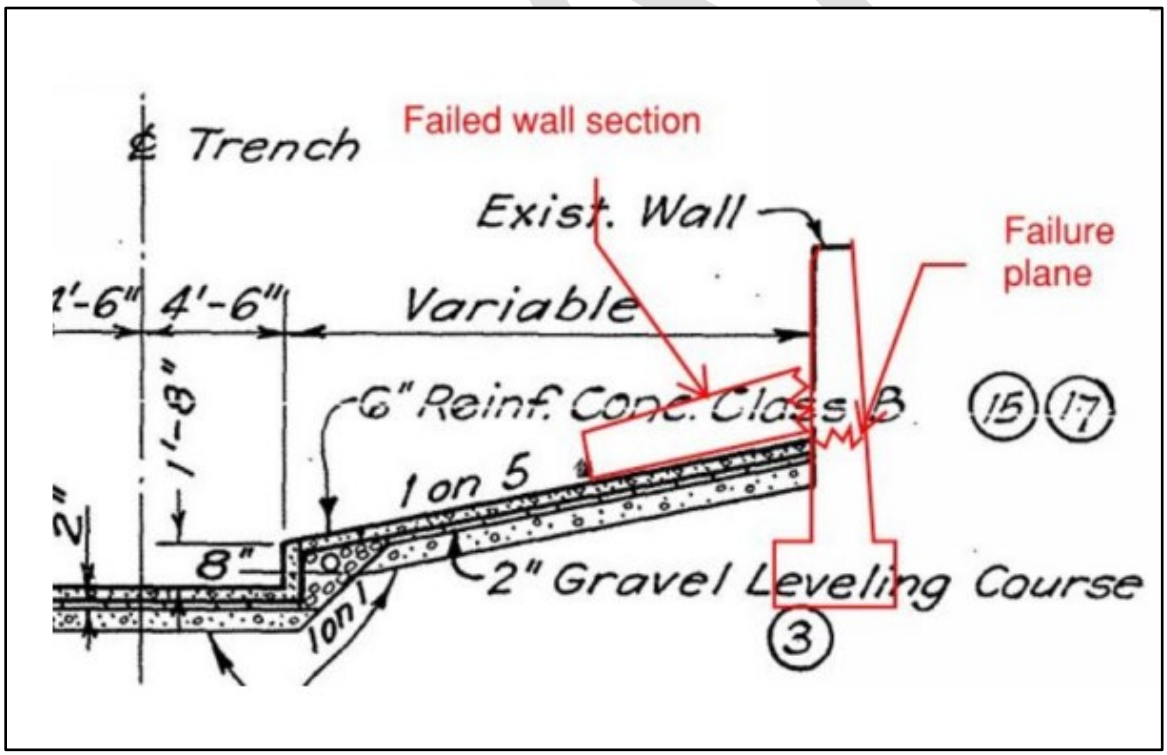


**Figure 6-5. Wall Failure Sketch**

**Figure 6-6. Wall Upstream of Otis Street**
Note: Notice previously added tiebacks to the wall. (See also Figure 6-8 for measurements of wall movements in this area.)

**Figure 6-7. Broken Drain behind the Wall Left Bank Downstream of Otis Street Bridge**

**WALLA WALLA COUNTY, WASHINGTON**
**MILL CREEK FLOOD CHANNEL WALL MEASUREMENTS**

| POINT NO. | DESCRIPTION | 2014 MEASUREMENTS (7/23) | | | 2015 MEASUREMENTS (9/16) | | | 2014 TO 2015 DIFFERENCES | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | NORTH | EAST | ELEVATION | NORTH | EAST | ELEVATION | DELTA NORTH | DELTA EAST | DELTA ELEVATION |
| 201 | WALL-TARGET | 199943.823 | 99994.323 | 5007.27 | 199943.832 | 99994.325 | 5007.28 | -0.009 | -0.003 | -0.005 |
| 202 | WALL-TARGET | 199973.635 | 100025.859 | 5007.67 | 199973.641 | 100025.854 | 5007.67 | -0.006 | 0.005 | -0.006 |
| 203 | WALL-TARGET | 200009.018 | 100064.692 | 5008.35 | 200009.018 | 100064.687 | 5008.36 | 0.000 | 0.005 | -0.008 |
| | | | | | | | | | | |
| 204 | WALL-TARGET | | | | 199790.117 | 99761.701 | 5005.46 | | | |

201-203 - Mini prism permanently attached to wall on south side between Otis Street and Merriam Street.
204 - Aluminum threaded plate on top of wall on north side between Otis Street and Park Street.

*Note: Coordinate values are assumes. Units: Feet*

**Figure 6-8. Wall Monitoring between Otis Street and Merriam Street**

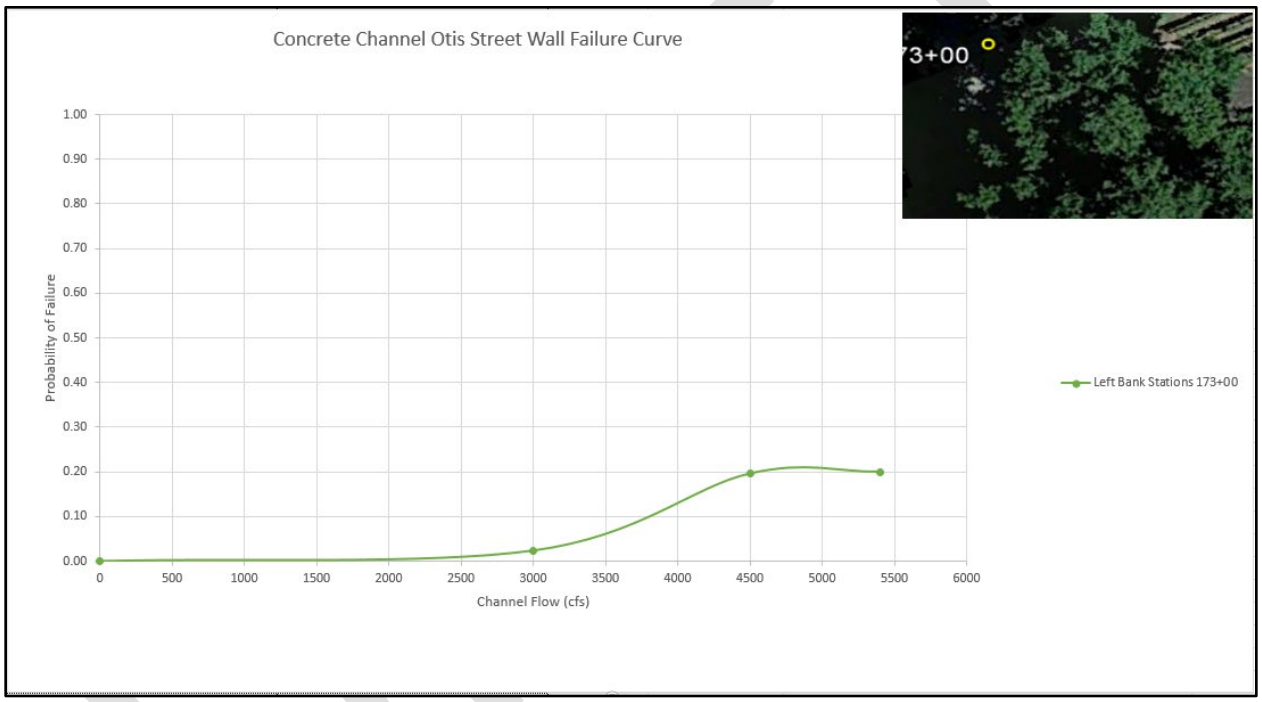


**Figure 6-9. Fragility Curve for a Channel Wall Failure Upstream of Otis Street Bridge**

## 6.4 CENTER COLUMN FAILURE ANALYSIS

During the channel inspection in the downtown area under the Die Brucke building it was noted that the center wall supporting the floor of the building above is actually composed of a series of pipe columns. These columns are not attached to the header beam above, and it is not known if the columns have any embedment into the concrete below. See Figures 6-10 through 6-12. There is evidence that they are only bearing on the concrete below (see the list of findings below). This center wall is in a curved portion of the channel, which leads to the main concern about the column support system. During high flow events, the water velocities were calculated to be up to 22 ft/s. The high flow velocity going around the corner in the channel leads to a significant super elevation of the flows in the channel. The super elevated water

applies a pressure on the wood wall due to the differential head. There is also a thrust component to this pressure due to turning or redirecting of the flow around the corner. The radius of the curve becomes smaller on the downstream end of the wall, increasing the loading in that range. For analysis, it was assumed that the only connection between the pipe columns and the concrete below was due to the friction between the concrete and the steel column. The capacity of the friction to resist the horizontal force is based on the vertical load on the columns from the floor above, and in some cases, the second floor and the roof. No drawings of the building above have been available for this analysis to help validate the loads on each one of these columns.

For this failure analysis, estimates were made for the floor loads in the building. High and low estimates were made to establish a variable range to calculate a probability of the capacity of the friction between the concrete and the pipe, failing to resist the demand of the horizontal pressure loads. The load cases shown below were developed from water surface elevations from the DM7 seen in Figure 6-11 for the 3,000 cfs, 4,500 cfs, and 5,400 cfs flow events. There is not a revised hydraulic model to validate the hydraulic conditions of this area in the channel. Therefore, it was recommended by hydraulic design that these flow conditions be used. The analysis does not consider any potential impact debris loads that could also be added to the hydraulic loads on this center wall. A lot of assumptions would have to be made to add debris impact such as the size of the debris, and how it was be applied to the wall. The hydraulic conditions can be readily calculated with some degree of certainty. See Annex B, Plate B-10 for a summary of loading diagrams for this failure.

It is also assumed that there would not need to be any additional failure probability added into the fragility curves to go from a failure to a blockage of the channel. In other words a failure probability of the pipe columns can be multiplied directly to economic consequences of a 90 percent channel blockage, resulting in the channel flows backing up and exiting the channel upstream at the Colville street area. Also it is assumed that the failure of any of the pipe columns supporting the floor of the building above could lead to a significant failure of the building.

Pipe column wall inspection findings:

- The pipe columns are 5 inch Double-Extra Strong pipe with 0.8 inch wall thickness based on one exposed column end.
- There are 15 pipe columns approximately 6.33 ft tall.
- Spacing of the pipe columns range from 8.85 ft to 10.7 ft.
- The top bearing plate is not welded to the top of the pipe column. The bearing plates are 1 inch thick.
- There is a wood spacer block and wood header beam on top of steel columns and bearing plates. Some of the wood spacer blocks have experienced significant crushing and deformation due to the load on top of the columns.

- Bottom condition of the column connection to the concrete footing below is not known. One column on the most upstream end of the column wall is not bearing on the concrete at least on one edge. Based on this condition it is assumed that the rest of the columns are likely only in bearing on the concrete.
- 2x12 wood planks on either side of pipe columns form a wall between the columns. Based on reaching into the void between the planks it appears to be filled with loose gravel. It is possible there is a small amount of cement in some areas.
- Ultrasonic thickness gauge was used unsuccessfully to measure the pipe wall thickness.

**Figure 6-10. Top Column Bearing Plate, No Welds or Bolts Connecting the Column to the Base Plate**

**Figure 6-11. Pipe Column Center Wall**

DRAFT

**Figure 6-12. Center Wall Column Wood Plank Connections**

**Table 6-2. Left Super Elevation Depth from Top of Right Wall per DM7 Drawing for Varying Channel Flows**

| Flow (cfs) | Depth (ft) |
|---|---|
| 5,400 | 2.08 |
| 4,500 | 2.67 |
| 3,000 | 3.58 |

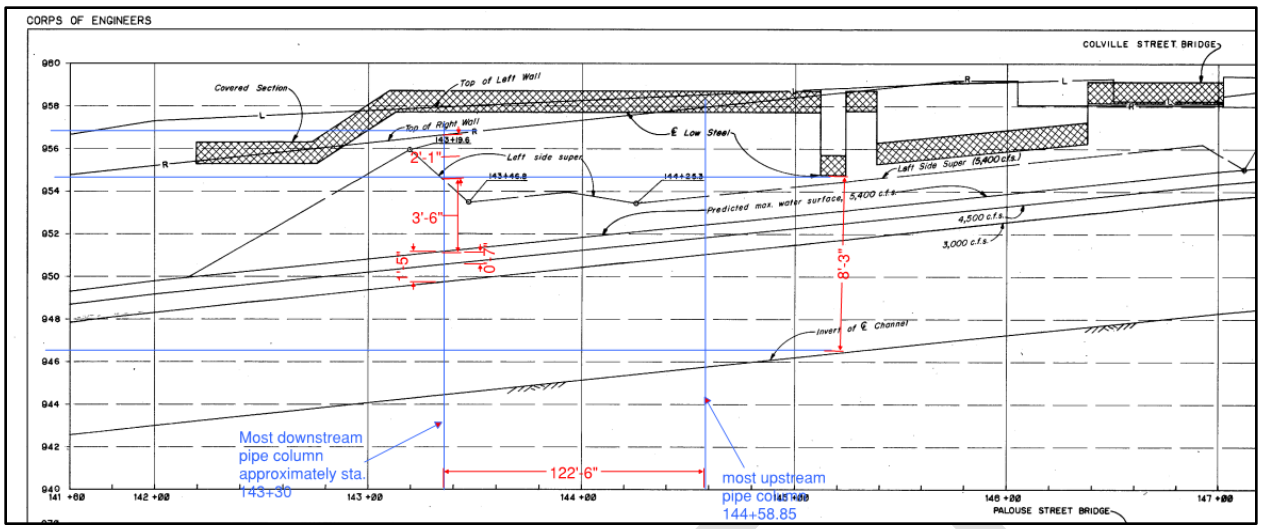


**Figure 6-13. Flow Profiles from DM7**

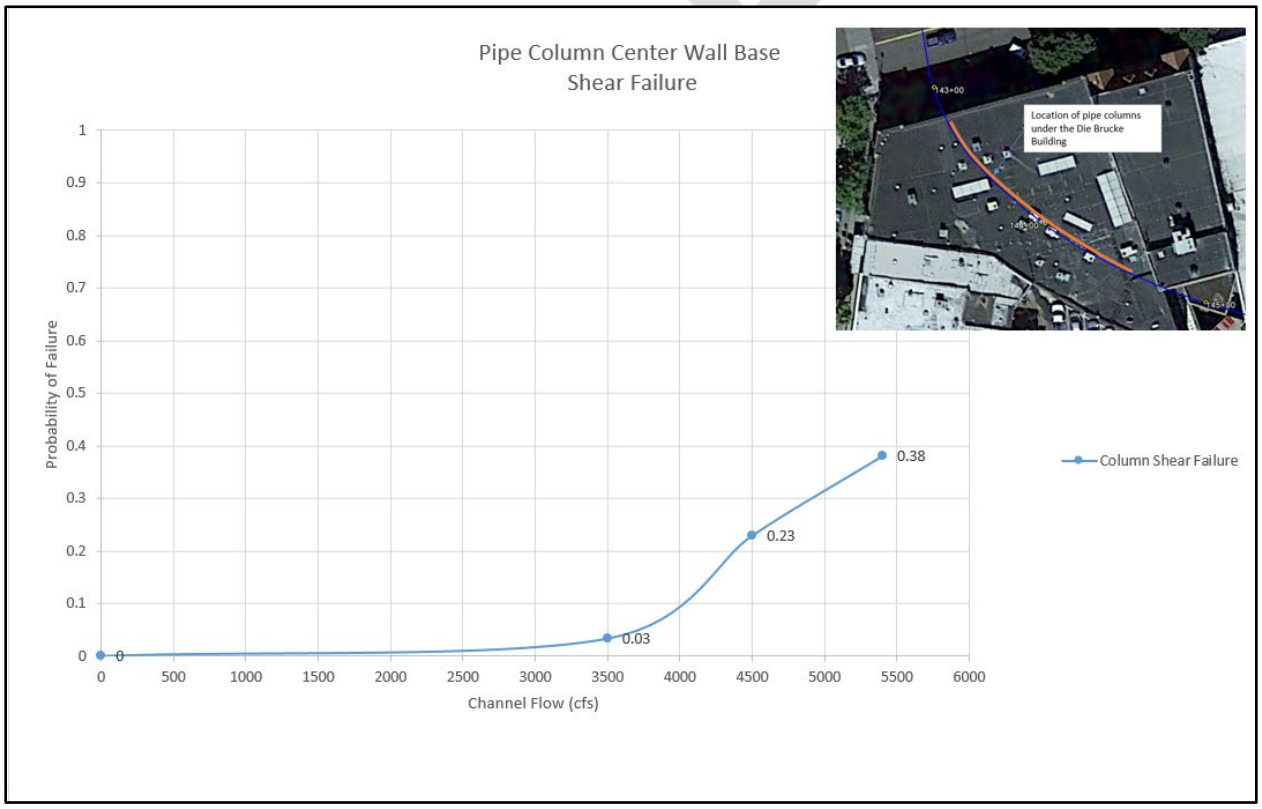


**Figure 6-14. Fragility Curve for the Center Wall Pipe Column Failure**

## 6.5 CEILING FAILURE

Ceiling spans of the covered portion of the concrete channel section have significant deterioration in two locations. These one of these locations at Station 135+00 is located in a parking lot. The area has been previously condemned for use and is fenced off. The second area is located at station 138+00 and is behind the Pantorium Building. This area is also blocked off to prevent parking on the ceiling span. These areas were noted in the Sargent 2016 engineering report as needing replacement or rehabilitation. It was assumed for this analysis that the initiation of failure at these locations could come from high water flows inducing vibrations, or from the water itself impacting the bottom of the ceiling during high flows. With the ceiling spans failed, the wall in that area of the channel would no longer be supported laterally. The walls in that area could also be damaged from the ceiling failure. The cantilever wall model that was used to analysis the walls for failure in other locations from a water surcharge from water getting behind the walls was used for this location. The fragility curve was developed from the wall model. The probability of failure for the station 135+00 location was used for this failure mode as it was slightly higher than the curve for the station 138+00 location. Also a failure of the ceiling at the 135+00 location leading to a blockage would also likely affect the 138+00 location. Using expert elicitation It was assumed very unlikely that the debris as a result of the covering collapse would block the channel, however if a blockage occurs it could create a worst case up to a 90 percent blockage. The 90 percent blockage of the channel in the covered portion would lead to the same hydraulic consequences as the center wall pipe column failure.

**Figure 6-15. Channel Ceiling Condition Station 135+00**

**Figure 6-15. Channel Ceiling Condition Station 138+00**

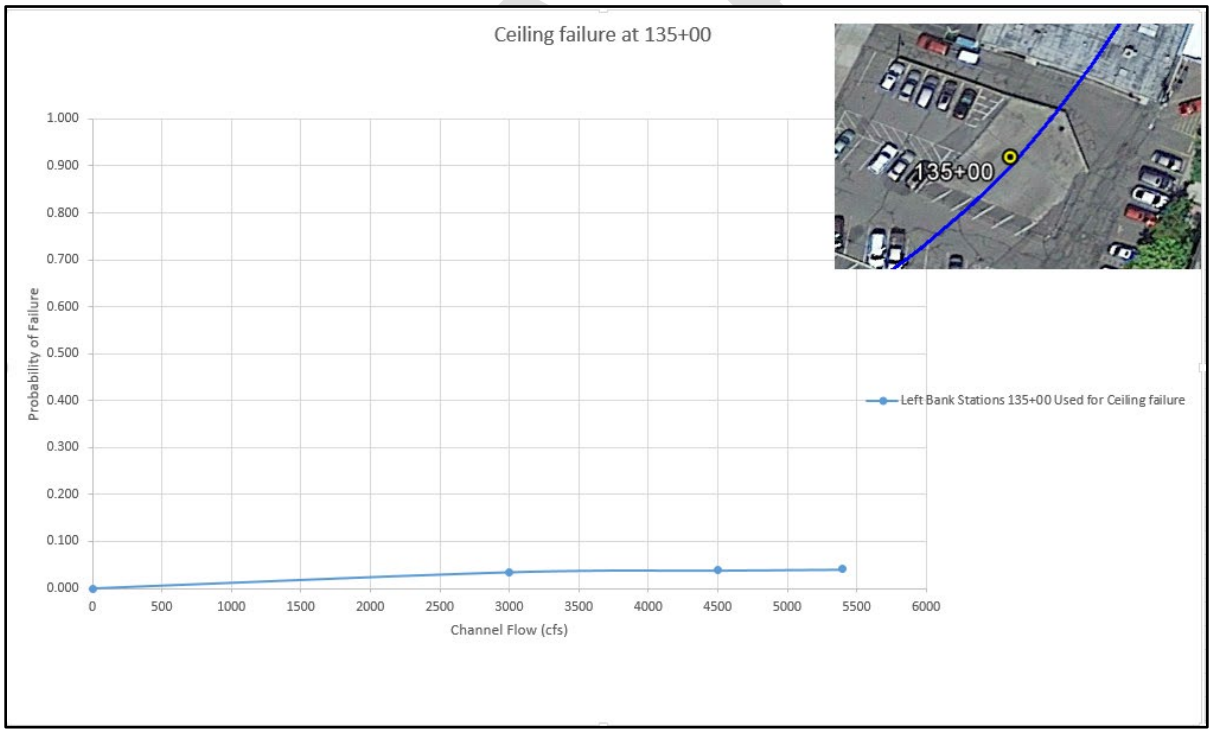


**Figure 6-16. Fragility Curve for the Channel Ceiling Failure**

## 6.6 CONCRETE CHANNEL REHABILITATION

For a summary of the recommended concrete channel rehabilitation see Section 3.1.3 in the main report, and Annex B Plates B-1 to B-8. Based on the cost of the rehab as listed in 3.1.3 the scope of the rehab was refined down to the most critical areas for the TSP, the reduced rehab is described in Section 3.1.9 of the main report. Based on analysis and past performance, there will be failures that will occur in the areas that will not be rehabbed in the TSP in the next 50 years. It is assumed that these failures will be localized failures that could occur but likely not lead to substantial consequences from channel blockage and flooding.

DRAFT

# SECTION 7 - Additional Requirements

## 7.1 CONSTRUCTION PROCEDURES AND WATER CONTROL PLAN

There are several locations where nearby water springs flow into the channel through cracks and joints in the concrete as well as from drains through the walls. Dewatering pumps will be required in the low flow channel to pump and lower the water a minimum of one foot below the work area. Past efforts to capture this water has been challenging as the water seeps into the work area from these flowing spring sources and lowering the local groundwater doesn't improve conditions. Temporary diversion of the channel flow around work areas will also be required.

## 7.2 FLOOD EMERGENCY PLANS FOR AREAS DOWNSTREAM OF CORPS DAMS

An Emergency Action Plan (EAP) with inundation maps exists for the diversion and storage dams and is routinely updated by USACE under the Dam Safety Program. The MCFCZD has emergency response procedures for their area of responsibility which are coordinated with County Emergency Management and USACE under the Levee Safety Program.

## 7.3 OPERATIONS AND MAINTENANCE

The sponsor (MCFCZD), in coordination with USACE, will continue to be responsible for annually traversing the entire length of the channel and inspecting the condition of the non-federal portion of the channel stabilizers, slope protection, levees, and concrete structures as well as addressing identified maintenance and repair needs. USACE will continue its current responsibilities for the federal O&M portion of the Project.

## 7.4 ACCESS ROADS

This project is located within the city of Walla Walla and in most cases it will be feasible to use the existing public city streets for transportation of miscellaneous construction equipment and hauling of excavated material, debris and construction materials.

## 7.5 CORROSION MITIGATION

Coatings and/or cathodic protection will be included in the design as required for materials which are installed in the soil.

## 7.6 COST ESTIMATES

A summary of the recommended plan costs is provided in Section 4.1.1 of the main report.

## 7.7 SCHEDULE FOR DESIGN AND CONSTRUCTION

This schedule for this study is provided in Section 4.2.2 of the main report.

DRAFT

## 7.8 PLATES, FIGURES, AND DRAWINGS

As-built and other project related drawings referenced are provided in Annex A. Current TSP concept drawings are shown in Annex B.

## 7.9 DATA MANAGEMENT

During the feasibility study, electronic data was compiled and maintained in project folders on the server for each discipline involved. This data is backed up regularly by USACE's data manager (ACE-IT).

## 7.10 USE OF METRIC SYSTEM MEASUREMENTS

The data used to produce the H&H models were all done in English units. Converting these data and drawings from English to Metric would have created additional work effort for the design team resulting in unnecessary schedule delays and labor costs.

DRAFT

This page intentionally left blank

DRAFT

# SECTION 8 - REFERENCES

Derkey, R.E, D.F. Stradling, K.A. Lindsey, and T.L. Tolan. 2006. Geologic Map of the College Place and Walla Walla 7.5-minute Quadrangles, Walla Walla County, Washington, and Umatilla County, Oregon. Washington Division of Geology and Earth Resources, Olympia, WA.

Duncan, J.M. 1999. *Factors of Safety and Reliability in Geotechnical Engineering*, ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 126, No. 4, pp. 307-316.

Harr, M.E. 1987. *Reliability-Based Design in Civil Engineering*, McGraw-Hill, 290 p.

Petersen, M. D., Moschetti, M. P., Powers, P. M., Mueller, C. S., Haller, K. M., Frankel, A. D., Zeng, Y., Rezaeian, S., Harmsen, S. C., Boyd, O. S., Field, N., Chen, R., Rukstales, K. S., Luco, N., Wheeler, R. L., Williams, R. A., and Olsen, A. H., (2015). *The 2014 United States National Seismic Hazard Model.* Earthquake Spectra: December 2015, Vol. 31, No. S1, pp. S1-S30.

Geo-Slope International, Ltd. 2019. *GeoStudio SLOPE/W*. Two-Dimensional Slope Stability Analyses Using the Limit Equilibrium Computer Program. Version 10.0.0.17401.

American Society of Civil Engineers (ASCE). 2017. *Minimum Design Loads and Associated Criteria for Buildings and Other Structures (7-16)*.

Anderson Perry and Associates, Inc. 2011. Levee Deficiency Correction Plan, Walla Walla County, Mill Creek Systems P and Q.

It Sargent Engineers, Inc. 2016. Inspection of Structures over Mill Creek. Prepared for the City of Walla Walla. Project Number: A15159.00.

U.S. Geological Survey. 2014. Seismic Hazard Curve for PGA. Retrieved from 2014 NSHM Gridded Data: https://www.usgs.gov/natural-hazards/earthquake-hazards

United States Army Corps of Engineers. 2003. Engineer Manual 1110-2-1902, *Engineering and Design: Slope Stability*. October.

United States Army Corps of Engineers. 2000. Engineer Manual 1110-2-1913, *Engineering and Design: Design and Construction of Levees*. April.

United States Army Corps of Engineers. 1994. Engineer Manual 1110-2-1601, *Engineering and Design: Hydraulic Design of Flood Control Channels*. June.

United States Army Corps of Engineers. 1989. Engineer Manual EM 1110-2-2502, *Retaining and Flood Walls*. September.

DRAFT

United States Army Corps of Engineers. 1999. Engineer Regulation 1110-2-1150, *Engineering and Design for Civil Works Projects*. August.

United States Army Corps of Engineers. 1997. Engineer Technical Letter 1110-2-547, *Engineering and Design: Introduction to Probability and Reliability Methods for Use in Geotechnical Engineering*. September.

United States Army Corps of Engineers, Seattle District. 1982. *The Geological and Seismological Review, Lower Snake River Dams, McNary and Mill Creek*.

United States Army Corps of Engineers, Walla Walla District. 1982. *Mill Creek Design Memorandum No. 7 – Main Channel Rehabilitation*. December.

United States Army Corps of Engineers. 2016. *Mill Creek Project (WA00348) Mill Creek, Walla Walla, Washington - Dam Safety Modification Study Existing Condition Risk Assessment* (FOUO/internal only). August.

United States Army Corps of Engineers. 2018. *Mill Creek Levee Systems (6005000009 & 6005000010) Semi-Quantitative Risk Assessment* (FOUO/internal only). May.

United States Army Corps of Engineers, Walla Walla District. 2018. *Periodic Inspection Report No. 12, Mill Creek Project (WA00348), Mill Creek, Washington*. June.

United States Army Corps of Engineers, Walla Walla District. 2018. *Continuing Eligibility Inspections of the Mill Creek Flood Risk Reduction Project*.

United States Army Corps of Engineers. Walla Walla District. 2019. *Mill Creek Expert Elicitation* (FOUO/internal only). April.

DRAFT