Summary of Attachments

Attachment 1 – Site Maps
- 1-1 – Location Map
- 1-2 – Taxlot Map
- 1-3 – ODFW Fish Distribution Map

Attachment 2 – Landowner Agreement Form(s)
- 2-1 – Landowner Agreement Form – Young Life
- 2-2 – SF 299 Application
- 2-3 – Wasco County Communications

Attachment 3 – Documentation of matching funds
- 3-1 – Match letter from Young Life

Attachment 4 – Project feasibility documentation
- 4-1 – Memorandum – Storage Increase Feasibility
- 4-2 – Site Suitability and Geotechnical Evaluation Report
- 4-3 – Hydrology and Inflow Design Storage Capacity and Breach Analysis Report

Attachment 5 – Letters (of support)
- 5-1 – Letter from Oregon Department of Fish and Wildlife
- 5-2 – Letter of support from Wasco County Planning Department

Attachment 6 – Engineering designs
- 6-1 – Engineering designs for dam raise and spillway reconstruction
- 6-2 – Engineering designs for fish passage at Muddy Creek falls

Attachment 7 – Storage permit
- 7-1 – Proposed Final Order (R-88276)
- 7-2 – Land Use Compatibility Statements (Jefferson and Wasco)

Attachment 8 – Detailed Budget
- 8-1 –Detailed Budget

Attachment 1

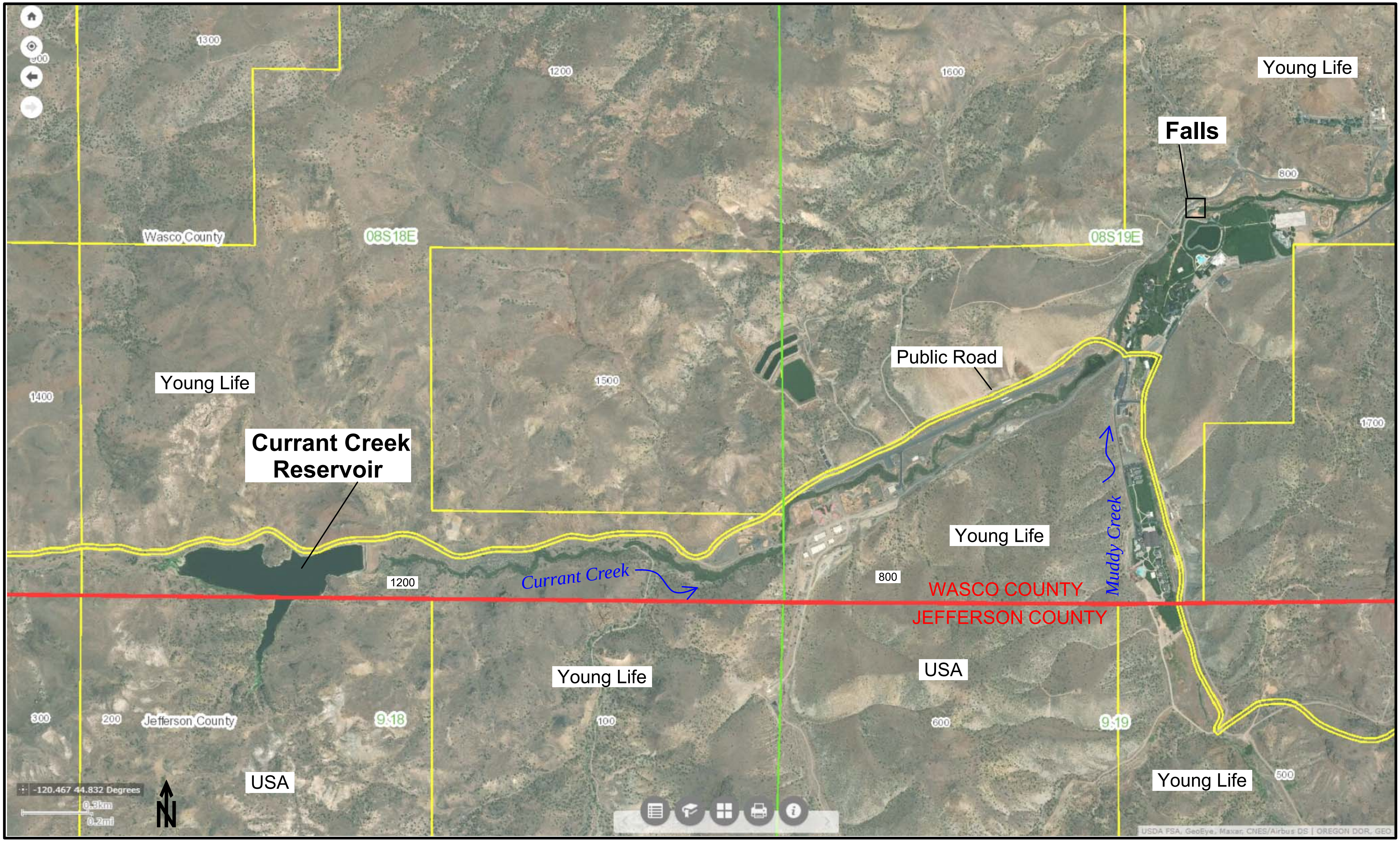
Site Maps

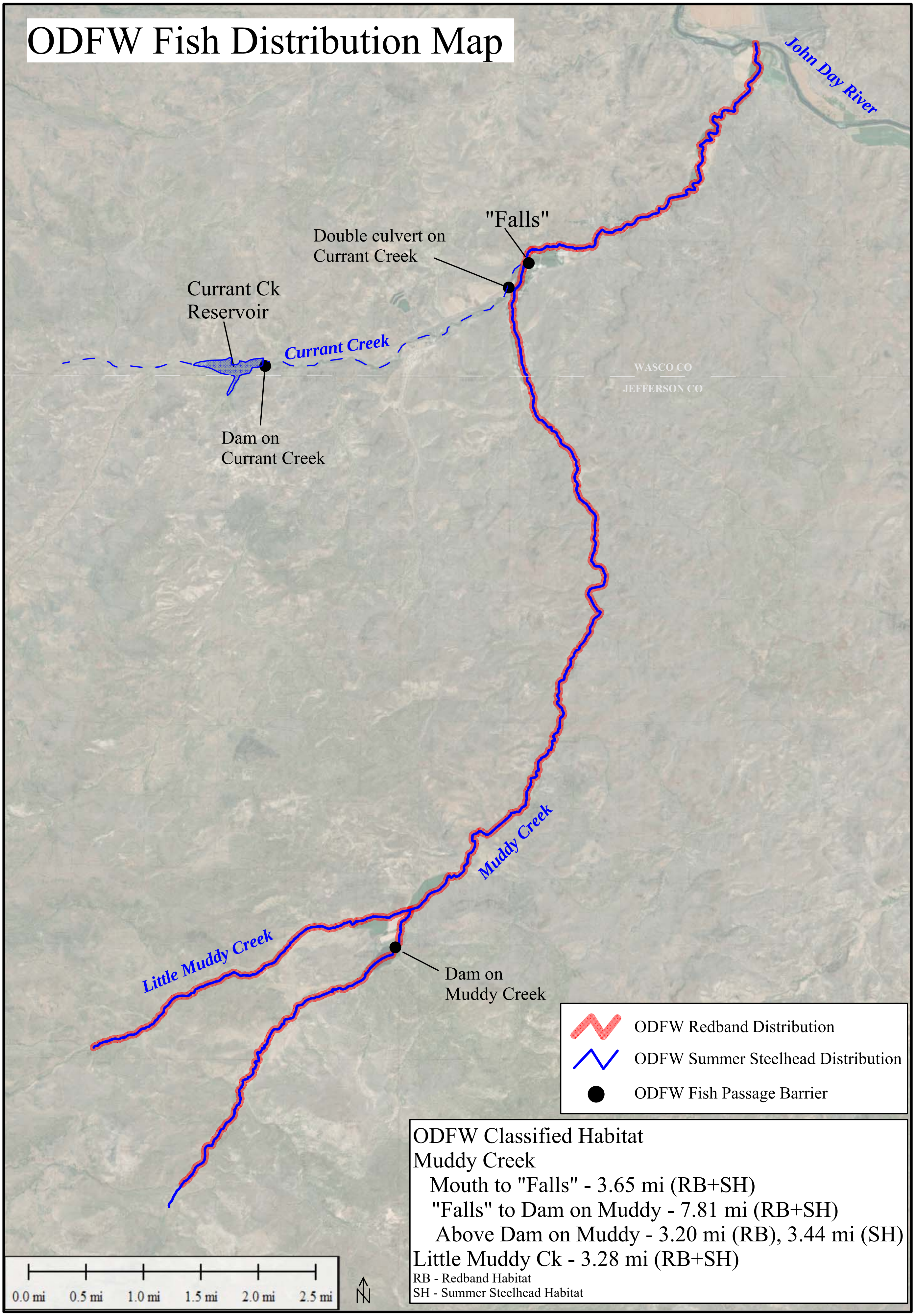
1-1 - Location Map (1p)
1-2 - Taxlot Map (1p)
1-3 - ODFW Fish Distribution Map (1p)

Attachment 1-1

Location Map

Taxlot Map


Young Life
Falls
1300
1200
1600
800
Wasco County
08S18E
08S19E
Public Road
Young Life
1400
1500
1700
Currant Creek Reservoir
Young Life
Currant Creek
1200
800
Muddy Creek
WASCO COUNTY
JEFFERSON COUNTY
Young Life
USA
300
200
Jefferson County
9.18
100
600
9.19
Young Life
500
USA
-120.467 44.832 Degrees
0.3km
0.2mi
N


ODFW Fish Distribution Map
John Day River
"Falls"
Double culvert on
Currant Creek
Currant Ck
Reservoir
Currant Creek
Dam on
Currant Creek
WASCO CO
JEFFERSON CO
Muddy Creek
Little Muddy Creek
Dam on
Muddy Creek
ODFW Redband Distribution
ODFW Summer Steelhead Distribution
ODFW Fish Passage Barrier
ODFW Classified Habitat
Muddy Creek
Mouth to "Falls" - 3.65 mi (RB+SH)
"Falls" to Dam on Muddy - 7.81 mi (RB+SH)
Above Dam on Muddy - 3.20 mi (RB), 3.44 mi (SH)
Little Muddy Ck - 3.28 mi (RB+SH)
RB - Redband Habitat
SH - Summer Steelhead Habitat
0.0 mi
0.5 mi
1.0 mi
1.5 mi
2.0 mi
2.5 mi
N

Attachment 2

Landowner Agreement Form(s)

2-1 - Landowner Agreement Form – Young Life (1p)
2-2 - SF 299 Application (23p)
2-3 - Wasco County Communications (1p)

OREGON
WATER RESOURCES DEPARTMENT

Water Project Grants and Loans

Landowner Agreement

Instructions to Applicants: Work with landowners to complete this form for all properties on which the proposed project would occur. Submit this completed form as part of your grant/loan application. For questions contact WRD_DL_waterprojects@oregon.gov.

Project and Applicant Information

Project Name: Currant Creek Storage, Creek Flow Augmentation and Fish Passage Project

Funding Applicant: Young Life Co-Applicant (if applicable): _____

Funding Applicant Contact Information:
Name: Chris Marshall
Phone Number: 503 487-7286
Email Address: cmarshall@sc.younglife.org

Co-Applicant Contact Information:
Name:_____
Phone Number: _____
Email Address: _____

Landowner Information

Landowner(s) Name: Young Life

Landowner Authorized Representative: Chris Marshall

Landowner Contact Information (or Authorized Representative)
Address: 13625 NW Cornell Road #200 (optional) Phone Number: 503 487-7286
(required) Portland, OR 97229 (optional) Email Address: cmarshall@sc.younglife.org

Property Information

List each property owned by the above-mentioned Landowner on which the project would occur:

County	Tax map	Lot number
Wasco	08S18E	1200
Wasco	08S19E	800

Landowner Acknowledgement

1. Young Life is/are the legal owner(s) (the Landowner) of the above described property (the Property).
2. I am authorized to act on behalf of the Landowner.
3. I am aware of and agree to the above-mentioned proposed project and grant permission for the Applicant, and the Applicant's agents, to conduct the following activities on the Property. (List activities below)

a. Raise Currant Creek Dam and reconstruct spillway with all associated infrastructure
b. Construct new stream channel at Muddy Creek Falls including culverts being replaced with a bridge
c. Install all necessary monitoring equipment

4. I am aware that monitoring information related to the Project is a matter of public record.
5. I certify that the above-mentioned information is true and accurate, I am aware of and agree to the proposed work, and I am authorized to sign as the Landowner or Authorized Representative.

Signature of Landowner or Authorized Representative:
Date: 5·28·2020 Print Name: Chris Marshall

STANDARD FORM 299 (05/2009)
Prescribed by DOI/USDA/DOT
P.L. 96-487 and Federal
Register Notice 5-22-95

APPLICATION FOR TRANSPORTATION AND UTILITY SYSTEMS AND FACILITIES ON FEDERAL LANDS

FORM APPROVED
OMB Control Number: 0596-0082
Expiration Date: 1/31/2017

FOR AGENCY USE ONLY
Application Number

Date Filed

NOTE: Before completing and filing the application, the applicant should completely review this package and schedule a preapplication meeting with representatives of the agency responsible for processing the application. Each agency may have specific and unique requirements to be met in preparing and processing the application. Many times, with the help of the agency representative, the application can be completed at the preapplication meeting.

1. Name and address of applicant (*include zip code*)

Young Life - Washington Family Ranch
P.O. Box 220
Antelope, OR 97001

2. Name, title, and address of authorized agent if different from item 1 (*include zip code*)

David Newton, Principal Geological Engineer/Engineering Geologist
Newton Consultants, Inc.
P.O. Box 1728, 1937 N Business 97

3. Telephone (area code)
(541) 489-3103
Applicant
Young Life-WFR
Authorized Agent
541-325-3905

4. As applicant are you? (*check one*)

a. ☐ Individual
b. ☒ Corporation*
c. ☐ Partnership/Association*
d. ☐ State Government/State Agency
e. ☐ Local Government
f. ☐ Federal Agency

** If checked, complete supplemental page*

5. Specify what application is for: (*check one*)

a. ☐ New authorization
b. ☐ Renewing existing authorization No.
c. ☐ Amend existing authorization No.
d. ☐ Assign existing authorization No.
e. ☐ Existing use for which no authorization has been received *
f. ☒ Other*

** If checked, provide details under item 7*

6. If an individual, or partnership are you a citizen(s) of the United States? ☐ Yes ☐ No

7. Project description (describe in detail): (a) Type of system or facility, *(e.g., canal, pipeline, road)*; (b) related structures and facilities; (c) physical specifications *(Length, width, grading, etc.)*; (d) term of years needed: (e) time of year of use or operation; (f) Volume or amount of product to be transported; (g) duration and timing of construction; and (h) temporary work areas needed for construction *(Attach additional sheets, if additional space is needed.)*

Young Life-Washington Family Ranch (YL-WFR), requests perpetual BLM right-of way grant or perpetual easement to inundate up to about 10.2 acres of United States (U.S.) owned land (Section 2, T9S, R18E). Site vicinity is shown on Fig 1. This U.S. owned land is landlocked inside YL-WFRl land with no public access. Inundation of about 5.7 acres presently occurs when existing Currant Creek Reservoir is full. Inundation of about 4.5 more acres will occur at full pool by raising existing crest of existing emergency spillway for more reservoir storage. Request is for renewal of prior authorization for current 5.7 acre-inundation and a new authorization for additional 4.5-acre inundation (total 10.2 ac). Spillway crest raise & related work to occur in existing spillway and possibly on the dam, all on WFR-YL land. Inundation areas are on Figure 3. Storage water now backs into gully draining U.S. land northward, to the reservoir. Added storage will back more water up the gully and cover small area at west end of reservoir. Dam and reservoir are on Currant Creek, tributary to Muddy Creek-John Day River. Normal spillway operation is during larger winter-early summer runoff flows. Crest raise includes earth embankment (or possible labyrinth weir), channel shaping, erosion provisions and reconstruct eroded parts of the spillway; about 90 days summer/fall work. Storage increase is up to 300 to 500 acre-feet in Currant Creek Reservoir for uses including: drinking/domestic supply, irrigation, including pollution abatement and creek flow augmentation to improve water supply for needed seasonal enhancement of downstream fish habitat below the falls at Young Life's Canyon Camp. See Supplemental Information & Figures 1, 2, 3.

8. Attach a map covering area and show location of project proposal

9. State or Local government approval: ☒ Attached ☐ Applied for ☐ Not Required

10. Nonreturnable application fee: ☒ Attached ☐ Not required

11. Does project cross international boundary or affect international waterways? ☐ Yes ☒ No *(if "yes," indicate on map)*

12. Give statement of your technical and financial capability to construct, operate, maintain, and terminate system for which authorization is being requested.

YL-WFR has maintained and monitored the Currant Creek Reservoir since obtaining the property in 1997. David Newton (consulting water resource engineer and engineering geologist), H. A. McCoy Engineering & Surveying, LLC will help develop the spillway construction plan to raise the crest, increase storage and make spillway repairs with related coordination with the Oregon State Dam Safety Engineer and will consult with YL -WFR on the construction work. These activities are necessary for public safety improvements and they will be performed in accordance with current Oregon Water Resources Department, Dam Safety Division regulatory standards, and in coordination with the Oregon State Dam Safety Engineer. YL-WFR has budgeted the funds to perform the engineering activities. The spillway component of the project must be completed and financial capability for this is and will be established.

13a. Describe other reasonable alternative routes and modes considered.

Other considered alternatives include relocating the emergency spillway to the north side of the dam with County road relocation/raise, over raised dam with dam overflow spillway, or dam raise with new spillway through a ridge south of the existing spillway with discharge into a natural drainage on U.S.-owned land.

b. Why were these alternatives not selected?

Alternatives were not chosen due to difficult construction, relocation and/or raising of the Wasco County road, larger scope of construction work and related costs for combined dam and spillway costs, which exceed the chosen alternative, and spillway discharge onto U.S.-owned lands.

c. Give explanation as t o why it is necessary to cross Federal Lands.

Water encroaches onto U.S.-owned land through a natural drainage gully extending north, from U.S. land to the reservoir. Constructing a dam across the gully to contain reservoir water on YL-WFR property would impound runoff from the U.S. land that otherwise drains naturally through the gully to reservoir; inundation of U.S. land is unavoidable.

14. List authorizations and pending applications filed for similar projects which may provide information to the authorizing agency. *(Specify number, date, code, or name)*

PLEASE SEE ATTACHED SUPPLEMENTAL INFORMATION

15. Provide statement of need for project, including the economic feasibility and items such as: (a) cost of proposal (construction, operation, and maintenance); (b) estimated cost of next best alternative; and (c) expected public benefits.

PLEASE SEE ATTACHED SUPPLEMENTAL INFORMATION

16. Describe probable effects on the population in the area, including the social and economic aspects, and the rural lifestyles.

PLEASE SEE ATTACHED SUPPLEMENTAL INFORMATION

17. Describe likely environmental effects that the proposed project will have on: (a) air quality; (b) visual impact; (c) surface and ground water quality and quantity; (d) the control or structural change on any stream or other body of water; (e) existing noise levels; and (f) the surface of the land, including vegetation, permafrost, soil, and soil stability.

PLEASE SEE ATTACHED SUPPLEMENTAL INFORMATION

18. Describe the probable effects that the proposed project will have on (a) populations of fish, plantlife, wildlife, and marine life, including threatened and endangered species; and (b) marine mammals, including hunting, capturing, collecting, or killing these animals.

PLEASE SEE ATTACHED SUPPLEMENTAL INFORMATION

19. State whether any hazardous material, as defined in this paragraph, will be used, produced, transported or stored on or within the right-of-way or any of the right-of-way facilities, or used in the construction, operation, maintenance or termination of the right-of-way or any of its facilities. "Hazardous material" means any substance, pollutant or contaminant that is listed as hazardous under the Comprehensive Environmental Response, Compensation, and Liability Act of 1980, as amended, 42 U.S.C. 9601 et seq., and its regulations. The definition of hazardous substances under CERCLA includes any "hazardous waste" as defined in the Resource Conservation and Recovery Act of 1976 (RCRA), as amended, 42 U.S.C. 6901 et seq., and its regulations. The term hazardous materials also includes any nuclear or byproduct material as defined by the Atomic Energy Act of 1954, as amended, 42 U.S.C. 2011 et seq. The term does not include petroleum, including crude oil or any fraction thereof that is not otherwise specifically listed or designated as a hazardous substance under CERClA Section 101(14), 42 U.S.C. 9601(14), nor does the term include natural gas.

Hazardous materials are not anticipated in this project.

20. Name all the Department(s)/Agency(ies) where this application is being filed.

Bureau of Land Management, Prineville

I HEREBY CERTIFY, That I am of legal age and authorized to do business in the State and that I have personally examined the information contained in the application and believe that the information submitted is correct to the best of my knowledge.

Signature of Applicant	Date

Title 18, U.S.C. Section 1001, makes it a crime for any person knowingly and willfully to make to any department or agency of the United States any false, fictitious, or fraudulent statements or representations as to any matter within its jurisdiction.

GENERAL INFORMATION
ALASKA NATIONAL INTEREST LANDS

This application will be used when applying for a right-of-way, permit, license, lease, or certificate for the use of Federal lands which lie within conservation system units and National Recreation or Conservation Areas as defined in the Alaska National Interest lands Conservation Act. Conservation system units include the National Park System, National Wildlife Refuge System, National Wild and Scenic Rivers System, National Trails System, National Wilderness Preservation System, and National Forest Monuments.

Transportation and utility systems and facility uses for which the application may be used are:

1. Canals, ditches, flumes, laterals, pipes, pipelines, tunnels, and other systems for the transportation of water.

2. Pipelines and other systems for the transportation of liquids other than water, including oil, natural gas, synthetic liquid and gaseous fuels, and any refined product produced therefrom.

3. Pipelines, slurry and emulsion systems, and conveyor belts for transportation of solid materials.

4. Systems for the transmission and distribution of electric energy.

5. Systems for transmission or reception of radio, television, telephone, telegraph, and other electronic signals, and other means of communications.

6. Improved right-of-way for snow machines, air cushion vehicles, and all-terrain vehicles.

7. Roads, highways, railroads, tunnels, tramways, airports, landing strips, docks, and other systems of general transportation.

This application must be filed simultaneously with each Federal department or agency requiring authorization to establish and operate your proposal.

In Alaska, the following agencies will help the applicant file an application and identify the other agencies the applicant should contact and possibly file with:

Department of Agriculture
Regional Forester, Forest Service (USFS)
Federal Office Building,
P.O. Box 21628
Juneau, Alaska 99802-1628
Telephone: (907) 586-7847 *(or a local Forest Service Office)*

Department of the Interior
Bureau of Indian Affairs (BIA)
Juneau Area Office
Federal Building Annex
9109 Mendenhall Mall Road, Suite 5
Juneau, Alaska 99802
Telephone: (907) 586-7177

Department of the Interior
Bureau of Land Management
222 West 7th Avenue
P.O. Box 13
Anchorage, Alaska 99513-7599
Telephone: (907) 271-5477 (or a local BLM Office)

U.S. Fish & Wildlife Service (FWS)
Office of the Regional Director
1011 East Tudor Road
Anchorage, Alaska 99503
Telephone: (907) 786-3440

National Park Service (NPA)
Alaska Regional Office, 2225
Gambell St., Rm. 107
Anchorage, Alaska 99502-2892
Telephone: (907) 786-3440

Note - Filings with any Interior agency may be filed with any office noted above or with the Office of the Secretary of the Interior, Regional Environmental Office, P.O. Box 120, 1675 C Street, Anchorage, Alaska 9513.

Department of Transportation
Federal Aviation Administration
Alaska Region AAL-4, 222 West 7th Ave., Box 14
Anchorage, Alaska 99513-7587
Telephone: (907) 271-5285

NOTE - The Department of Transportation has established the above central filing point for agencies within that Department. Affected agencies are: Federal Aviation Administration (FAA), Coast Guard (USCG), Federal Highway Administration (FHWA), Federal Railroad Administration (FRA).

OTHER THAN ALASKA NATIONAL INTEREST LANDS

Use of this form is not limited to National Interest Conservation Lands of Alaska.

Individual department/agencies may authorize the use of this form by applicants for transportation and utility systems and facilities on other Federal lands outside those areas described above.

For proposals located outside of Alaska, applications will be filed at the local agency office or at a location specified by the responsible Federal agency.

SPECIFIC INSTRUCTIONS
(Items not listed are self-explanatory)

7 Attach preliminary site and facility construction plans. The responsible agency will provide instructions whenever specific plans are required.

8 Generally, the map must show the section(s), township(s), and range(s) within which the project is to be located. Show the proposed location of the project on the map as accurately as possible. Some agencies require detailed survey maps. The responsible agency will provide additional instructions.

9 , 10, and 12 The responsible agency will provide additional instructions.

13 Providing information on alternate routes and modes in as much detail as possible, discussing why certain routes or modes were rejected and why it is necessary to cross Federal lands will assist the agency(ies) in processing your application and reaching a final decision. Include only reasonable alternate routes and modes as related to current technology and economics.

14 The responsible agency will provide instructions.

15 Generally, a simple statement of the purpose of the proposal will be sufficient. However, major proposals located in critical or sensitive areas may require a full analysis with additional specific information. The responsible agency will provide additional instructions.

16 through 19 Providing this information is as much detail as possible will assist the Federal agency(ies) in processing the application and reaching a decision. When completing these items, you should use a sound judgment in furnishing relevant information. For example, if the project is not near a stream or other body of water, do not address this subject. The responsible agency will provide additional instructions.

Application must be signed by the applicant or applicant's authorized representative.

EFFECT OF NOT PROVIDING INFORMATION: Disclosure of the information is voluntary. If all the information is not provided, the application may be rejected.

DATA COLLECTION STATEMENT

The Federal agencies collect this information from applicants requesting right-of-way, permit, license, lease, or certification for the use of Federal lands. The Federal agencies use this information to evaluate the applicant's proposal. The public is obligated to submit this form if they wish to obtain permission to use Federal lands.

Attachment 2.2

SUPPLEMENTAL

NOTE: The responsible agency(ies) will provide instructions	CHECK APPROPRIATE BLOCK	
I - PRIVATE CORPORATIONS	ATTACHED	FILED*
a. Articles of Incorporation	☐	☒
b. Corporation Bylaws	☐	☒
c. A certification from the State showing the corporation is in good standing and is entitled to operate within the State	☐	☒
d Copy of resolution authorizing filing	☐	☒
e. The name and address of each shareholder owning 3 percent or more of the shares, together with the number and percentage of any class of voting shares of the entity which such shareholder is authorized to vote and the name and address of each affiliate of the entity together with, in the case of an affiliate controlled by the entity, the number of shares and the percentage of any class of voting stock of that affiliate owned, directly or indirectly, by that entity, and in the case of an affiliate which controls that entity, the number of shares and the percentage of any class of voting stock of that entity owned, directly or indirectly, by the affiliate.	☐	☐
f. If application is for an oil or gas pipeline, describe any related right- of-way or temporary use permit applications, and identify previous applications.	☐	☐
g. If application is for an oil and gas pipeline, identify all Federal lands by agency impacted by proposal.	☐	☐
II - PUBLIC CORPORATIONS		
a. Copy of law forming corporation	☐	☐
b. Proof of organization	☐	☐
c. Copy of Bylaws	☐	☐
d. Copy of resolution authorizing filing	☐	☐
e. If application is for an oil or gas pipeline, provide information required by item "I - f" and "I - g" above.	☐	☐
III - PARTNERSHIP OR OTHER UNINCORPORATED ENTITY		
a. Articles of association, if any	☐	☐
b. If one partner is authorized to sign, resolution authorizing action is	☐	☐
c. Name and address of each participant, partner, association, or other	☐	☐
d. If application is for an oil or gas pipeline, provide information required by item "I - f" and "I - g" above.	☐	☐

*****If the required information is already filed with the agency processing this application and is current, check block entitled "Filed." Provide the file identification information *(e.g., number, date, code, name)*. If not on file or current, attach the requested information.

NOTICES

Note: This applies to the Department of Agriculture/Forest Service (FS)

This information is needed by the Forest Service to evaluate the requests to use National Forest System lands and manage those lands to protect natural resources, administer the use, and ensure public health and safety. This information is required to obtain or retain a benefit. The authority for that requirement is provided by the Organic Act of 1897 and the Federal Land Policy and Management Act of 1976, which authorize the secretary of Agriculture to promulgate rules and regulations for authorizing and managing National Forest System lands. These statutes, along with the Term Permit Act, National Forest Ski Area Permit Act, Granger-Thye Act, Mineral Leasing Act, Alaska Term Permit Act, Act of September 3, 1954, Wilderness Act, National Forest Roads and Trails Act, Act of November 16, 1973, Archeological Resources Protection Act, and Alaska National Interest Lands Conservation Act, authorize the Secretary of Agriculture to issue authorizations or the use and occupancy of National Forest System lands. The Secretary of Agriculture's regulations at 36 CFR Part 251, Subpart B, establish procedures for issuing those authorizations.

BURDEN AND NONDISCRIMINATION STATEMENTS

According to the Paperwork Reduction Act of 1995, an agency may not conduct or sponsor, and a person is not required to respond to a collection of information unless it displays a valid OMB control number. The valid OMB control number for this information collection is 0596-0082. The time required to complete this information collection is estimated to average 8 hours hours per response, including the time for reviewing instructions, searching existing data sources, gathering and maintaining the data needed, and completing and reviewing the collection of information.

The U.S. Department of Agriculture (USDA) prohibits discrimination in all its programs and activities on the basis of race, color, national origin, age, disability, and where applicable, sex, marital status, familial status, parental status, religion, sexual orientation, genetic information, political beliefs, reprisal, or because all or part of an individual’s income is derived from any public assistance. (Not all prohibited bases apply to all programs.) Persons with disabilities who require alternative means for communication of program information (Braille, large print, audiotape, etc.) should contact USDA’s TARGET Center at 202-720- 2600 (voice and TDD).

To file a complaint of discrimination, write USDA, Director, Office of Civil Rights, 1400 Independence Avenue, SW, Washington, DC 20250-9410 or call toll free (866) 632-9992 (voice). TDD users can contact USDA through local relay or the Federal relay at (800) 877-8339 (TDD) or (866) 377-8642 (relay voice). USDA is an equal opportunity provider and employer.

The Privacy Act of 1974 (5 U.S.C. 552a) and the Freedom of Information Act (5 U.S.C. 552) govern the confidentiality to be provided for information received by the Forest Service.

SUMMARY OF AMENDED SF-299 APPLICATION
YOUNG LIFE - WASHINGTON FAMILY RANCH
February 25, 2019

INTRODUCTION

Planned expansion of storage in the Currant Creek reservoir, owned by the Young Life – Washington Family Ranch (YL-WFR), requires an SF-299 permit for inundation of a relatively small area of land owned by the United States (U.S.). The land subject to inundation is within Section 2, T9S., R.18E., W.M. This entire land section is landlocked wholly within land owned by the YL-WFR and has no public access. Figure 1 (Vicinity Map) shows the general project location.

A SF-299 application was submitted to the Bureau of Land Management (BLM), Pineville, on August 16, 2017. That application has since been amended due to simplification of the project and need to renew a prior authorization for current inundation by the existing reservoir of about 5.7 acres of U.S. land and need for a new authorization to inundate an additional area of U.S. land estimated at 4.5 acres. The total area of inundation would be about 10.2 acres when the reservoir is full to a maximum potential reservoir water surface elevation at 1,845 feet.

The original and amended SF-299 applications are summarized below. More detailed technical descriptions are within the amended SF-299 application and are highlighted in yellow in the Supplemental Information document.

PURPOSE OF SF-299 APPLICATION

The SF-299 application is for permission to inundate a small area of land owned by the U.S. with waters of Currant Creek reservoir. The existing dam across Currant Creek is on YL-WFR land, creating the existing water reservoir. As the reservoir fills, water backs into a tributary drainage gully, draining northward from U.S.-owned lands to the reservoir. Currently, when the reservoir is full to the maximum operating water level elevation at 1,831 feet, about 5.7 acres of U.S.-owned land in the gully is covered with water. When the reservoir storage capacity is increased as currently proposed, up to about 4.5 acres of additional U.S.-owned land will be covered when the reservoir is full to a maximum potential water surface elevation of 1,845 feet. This acreage includes the original estimated 5.7 acres under existing full reservoir conditions and about 4.5 additional acres for the planned condition. The total area of inundation would be about 10.2 acres when the reservoir is full. Permission through the SF-299 application process is required for the land inundation.

ORIGINAL 2017 SF-299 APPLICATION

The original SF-299 application was based on a larger project which included raising the existing dam by 5 to 20 feet, a maximum reservoir water surface elevation adjusted upward to 1,855 feet, extension of the dam from its current south abutment across the existing emergency spillway, southward to a new abutment on a ridge, and a new emergency spillway that would discharge water from the ridge top onto U.S.-owned lands. Spillway discharge was originally planned to be conveyed through an existing natural drainage gully on U.S. lands back to Currant Creek, downstream of the dam.

Reservoir storage capacity would be increased by about 760 acre-feet. The project included potential rip rap and erosion control measures on U.S.-owned land. This project would also cover up to about 13 acres (existing inundation area of about 5 acres plus an additional area of about 8 acres from the proposed project) of U.S.-owned lands with water. The enclosed Figure 2 illustrates the original planned project.

AMENDED SF-299 APPLICATION

The enclosed amended SF-299 application is strictly for renewal of a prior authorization to inundate about 5.7 acres of U.S. land and for a new authorization to inundate an additional estimated 4.5 acres of U.S. land. The total area of U.S. land inundation is estimated at 10.2 acres. These areas of inundation are based on topographic surveys subsequent to the original SF-299 application.

The amended SF-299 application is based on a simplified project with all construction on YL-WFR land and no spillway discharge onto U.S.-owned land. Additional storage of up to 400 to 500 acre-feet will be achieved by raising the crest of the existing emergency spillway. Some minor raising of the existing dam (approximately 1 to 3 feet) is under consideration. All construction is on YL-WFR-owned land. The only impact to U.S.-owned land is inundation by reservoir water for up to about 10.2 acres. This area is in the afore-mentioned gully. Figure 3 illustrates the amended project location.

SUPPLEMENTAL INFORMATION FOR SF 299
APPLICATION FOR TRANSPORTATION AND UTILITY SYSTEMS AND FACILITIES ON FEDERAL LANDS
RIGHT-OF-WAY OR PERPETUAL EASEMENT TO INUNDATE UP TO 13 ACRES OF FEDERAL LAND MANAGED BY BLM
YOUNG LIFE – WASHINGTON FAMILY RANCH – CURRANT CREEK DAM
JEFFERSON COUNTY, OREGON

AMENDED

February 25, 2019

NOTE: Amendments to the original submitted SF-299 Application are highlighted in yellow.

PURPOSE OF AMENDMENTS TO SF-299 APPLICATION SUBMITTED TO BLM AUGUST 16, 2017

Introduction

Young Life – Washington Family Ranch (YL-WFR) plans to increase storage in its existing Currant Creek reservoir by raising the crest of the existing emergency spillway. The spillway modification will also account for repair needs in the existing spillway channel. Construction will all be on YL-WFR property. The general site location is shown on Figure 1 (Vicinity Map).

A small area of land owned by the United States will be inundated by reservoir water. This land is in Section 13, T9S., R18E., W.M. which is landlocked within YL-WFR property and has no public access. Part of this area is now inundated by reservoir water. A SF-299 permit application process is required for authorization to inundate this area. A SF-299 application was submitted to the BLM on August 16, 2017. Since then, project modifications have been made. Accordingly, an amended SF-299 application, of which this document is a part, is submitted herewith, to account for the modifications.

Purpose of Amended SF-299 Application

This amended SF-299 application is for renewal of a prior authorized inundation of about 5.7 acres which are inundated at full pool in the existing Currant Creek reservoir, and for a new authorization to inundate up to about 4.5 additional acres of U.S. owned land at full reservoir pool after the spillway crest is raised, as described below. The total area of inundated land is up to about 10.2 acres. No construction will occur on land owned by the U.S.

Background

When the original dam and reservoir were constructed circa 1983-84, an SF-299 permit was issued by the BLM to allow inundation of land owned by the U.S. This land is within a drainage gully that drains northward, from U.S.-owned lands, to the reservoir. Water from the reservoir backs into this gully, onto lands owned by the U.S. The area of inundation at full reservoir pool is approximately 5.7 acres. This original permit is not in effect after the land owners at that time abandoned the property. The planned raise of the spillway crest and related increase in reservoir storage will inundate an additional approximate area of about 4.5 acres of U.S. owned land. This additional land inundation at full reservoir pool would occur in the afore-mentioned gully and at the west end of the reservoir.

Since the original SF-299 application was submitted to the BLM on August 16, 2017, detailed topographic surveys and significant engineering evaluations have been completed that reduced the scope of the project. The scope of the project for the August 16, 2017 application included construction of an extended dam and new emergency spillway that would discharge onto United States (U.S.) owned land and inundate about 13 acres of U.S. owned land. This concept is illustrated on Figure 2. The amended project inundates up to about 10.2 acres of U.S. land with no spillway discharge onto U.S. owned land. This concept is illustrated on Figure 3.

Completed engineering work includes a hydrologic analysis conducted by West Consultants, Inc., which updated the design inflow rate for the existing emergency spillway to 11,900 cubic feet per second (cfs). This analysis was based on updated knowledge, technology and methods since the hydrologic analysis for the original design of the existing emergency spillway at the Currant Creek dam. The existing emergency spillway was designed for the 0.5 General Storm Probable Maximum Flood (PMF) inflow based on a 1982 analysis, data and technology available at that time. The inflow rate for which the existing spillway was originally designed and constructed is 22,400 cfs.

Amended Construction Project

The updated inflow rate for the spillway is significantly lower than the original rate, resulting in opportunities to reconstruct the emergency spillway with a higher crest elevation, bringing opportunities to increase storage in the reservoir. The additional storage provides YL-WFR with improved water supply reliability and additional supply for current and future water needs.

Initial project concepts presented in the original SF-299 application, including a new spillway with discharge onto U.S. owned lands, and are now less desirable considering that a new, less extensive and costly solution is planned. This new concept is the basis for amending the original SF-299 and consists of raising the crest of the existing spillway to increase storage and respond to YL-WFR water supply concerns over the longer term. Raising the crest of the existing spillway necessarily results in channel reconstruction below the crest, which responds to erosion repair needs. All construction will be on land owned by YL-WFR. No construction will be on U.S. owned land.

BLOCK 1 - 7. See SF 299 form for Young Life – Washington Family Ranch (YL-WFR).

BLOCK 7. Project Description

7.(a) Type of system or facility: Increase water storage for water supply through reconstructed emergency spillway crest. Reconstruction of the emergency spillway as described below will also bring it to current standards of the Oregon Water Resources Department, Dam Safety Division. All of the planned construction is on YL-WFR property. National Inventory of Dams # OR 00696, State file # C-64, OWRD water right permit R-8452.

The emergency spillway is located south of the current dam (Figure 1) and was designed with a capacity to pass the General Storm Probable Maximum Flood (PMF) of 22,400 cfs. Recently completed hydrologic analyses to update the inflow PMF according to current and upgraded technology revealed an inflow flood flow rate of 11,900 cfs. This flow is substantially lower than the 1982 estimate. This reduces the required flow capacity of the spillway and its related geometric requirements, providing a situation where the crest can be raised to provide additional storage for YL-WFR for improved supply reliability and ability to meet its future needs. Repairs of the erosional downcutting in the existing spillway are accounted for in the construction to raise the crest for storage.

A water storage permit application is being processed by the Oregon Water Resources Department (OWRD). This application is for the additional water storage that will result from construction of the project.

7.(b) Related structures and facilities: Currant Creek Dam. The dam is an engineered compacted earth fill structure approximately 70 feet high with a top width of 20 feet. . The primary reservoir outlet works through the dam consists of an existing concrete-encased steel pipe conduit measuring 30 inches across located near the north dam abutment. Operation is by a hand control wheel near the top of the dam. Engineered plans for the project will include installation of a new control valve at the downstream end of the pipe outlet cto improve operation and outlet water control. An existing powerline, serving only the applicant and no longer needed by the applicant will be removed. Figure 1 shows project area and general features.

7.(c) Physical specifications:

Inundation of Land Owned by the United States – Purpose of Amended SF-299 Application

Inundation of U.S. owned land occurs when the existing reservoir behind Currant Creek Dam is full. The area of inundation is about 5.7 acres. The U.S. owned land subject to this inundation is within Section 2, T9S., R18E., W.M.. Section 2 is wholly landlocked within land owned by YL-WFR, with no public access.

Inundation of U.S. owned land under present full reservoir conditions occurs in a natural drainage gully that conveys runoff from U.S. land to the reservoir. The gully forms a reservoir arm, extending south about 1,450 feet into U.S. owned land (see Figure 3) when the reservoir is full. The width of the inundation area at the inlet to this arm of the reservoir is roughly 250 feet, tapering to zero feet at the southern terminus of inundation. The surface area of the inundation is about 5.7 acres based on topographic surveys (Figure 3). The existing inundation condition was previously authorized for a prior land owner when the dam was constructed. This amended SF-299 application is to renew this prior authorization.

Raising of the spillway crest and added reservoir storage will increase the area of U.S. land inundated during full reservoir pool conditions. Inundation of additional U.S. land in the southerly extending arm of the reservoir will be up to about 4.5 acres. The width of the inundation area at the inlet of this arm of the reservoir will be roughly 500 feet, tapering to zero feet at the southern terminus of inundation. The inundation area will extend southward about 1,700 feet into U.S. owned land, where the gully divides into two forks.

This amended SF-299 application is to obtain a new authorization for this additional area of U.S. land inundation at full reservoir pool. The total area of U.S. land inundation for renewed authorization and the new authorization is up to about 10.2 acres. The total estimated area of inundation is shown on Figure 2 at 12.46 acres, which is based on the original proposed maximum reservoir water surface elevation of 1,855 feet. The maximum reservoir water surface elevation for the amended SF-299 is in the range of 1,838 to a maximum of 1,845 feet.

<u>Project Construction Elements on Private Land</u>

The current emergency spillway consists of a concrete control section at the inlet and a downstream channel originally excavated into volcanic rock. The channel at the concrete control section is about 180 feet wide with a surface elevation of 1,831 feet. Existing conditions are shown on Figure 3. The control section will be reconstructed with a new crest elevation of up to an approximate elevation of 1838 to 1,842 feet to increase storage capacity of the reservoir. The actual elevation will be confirmed through the engineering and design work for the new crest and whether a minor raise of 1 to 3 feet will be done to the existing dam. The downstream slope of the spillway channel from the crest will be re-engineered and reconstructed in response to the new crest elevation, which will account for repairs needed for the existing channel erosion.

Storage of the additional water resulting from the proposed improvements will be in the range of 300 to about 500 acre-feet, depending on the confirmed spillway crest elevation according to depth of spillway design flow and allowing for 2 feet of freeboard between the dam top and reservoir water surface during the design flood event. This will inundate a narrow, northward draining gully extending from U.S. land to the existing reservoir as shown on Figure 3. This natural drainage channel conveys runoff from the U.S. land into the reservoir. At full pool for the existing reservoir, about 5.7 acres of U.S. land in the gully is inundated. At the maximum potential full reservoir pool elevation of 1,845 feet after the spillway crest is raised, about 10.2 acres of U.S. land in the gully will be inundated. This area includes the inundation area for existing full reservoir conditions.

7.(d) Term of years needed: Perpetual use is requested.

7.(e) Time of year of use or operation: Use of water in Currant Creek Reservoir is year-round. Water use increases in spring and early summer, reaching peak use rates during July through August, then declines generally during September through October to the winter period of lowest use. The maximum elevation of the normal operating pool will be generally during late winter and spring months. Normal operation of the emergency spillway is during larger winter-spring flood runoff events.

Proposed creek flow enhancements through water releases from the reservoir will be scheduled according to input from the Oregon Department of Fish and Wildlife (ODFW) and National Oceanic and Atmospheric Administration (NOAA).

7.(f) Volume or amount of product to be transported: The amount of water to be stored is up to approximately 300 to 500 acre feet. No water will be transported. Water for storage in the reservoir is runoff from the Currant Creek watershed. Volume is seasonal with majority of fill occurring in the late winter, early spring months annually.

7.(g) Duration and timing of construction: The project is scheduled to begin in late summer/fall of 2019, or summer/fall of 2020, roughly over a 3 to 4 month period.

7.(h) Temporary work areas needed for construction: No construction is planned on lands owned by the United States. Very little impact to United States land is anticipated by up to 10.2 acres of inundation along the perimeter of an existing southward extending finger of the reservoir when the reservoir is full to a water surface elevation of 1,845 feet. Construction work will occur only on the YL-WFR property.

7.(i) Additional impacts: No impacts other than inundation of up to approximately 10.2 acres will occur on lands owned by the United States when the reservoir is full to a maximum water surface elevation of 1,845 feet.

No long term negative impacts to surrounding property owners are anticipated. There will be additional traffic in the project area limited to the construction period.

BLOCK 8 - 13. See SF 299 form for YL-WFR.

BLOCK 14. List authorizations and pending applications filed for similar projects which may provide information to the authorizing agency.
1) Land Use Compatibility Statement by:
Jefferson County Community Development Department
Jeff Spencer, Planning Director
85 SE "D" Street
Madras, OR 97741
Phone: 541.475.4462 E-mail: Jefferson.Spencer@co.jefferson.or.us

2) Land Use Compatibility Statement by:
Wasco County Planning Department
Attn: Dawn Baird
2705 East Second Street
The Dalles, OR 97058
Phone: (541) 506-2560 E-mail: wcplanning@co.wasco.or.us

3) Storage Permit Application by:
Oregon Water Resources Department
Attn: Lisa Graham, Water Right Application Caseworker
725 Summer Street NE, Suite A
Salem, Oregon 97301
(503) 986-0808 E-mail: Elisabeth.A.Graham@oregon.gov
4) Road Modification Approval by:
Wasco County Road Department
2705 East Second Street
The Dalles, OR 97058
(541) 296-5491

BLOCK 15. Provide statement of need for project, including the economic feasibility and items such as: (a) cost of proposal (construction, operation, and maintenance); (b) estimated cost of next best alternative; and (c) expected public benefits.

Project Need

The project need is to increase storage in the reservoir through construction of a higher spillway crest and bring the eroded emergency spillway at Currant Creek Dam up to performance and safety requirements of the Oregon Water Resources Department, Dam Safety Division according to the updated hydrologic analyses and updated inflow design flow for the PMF of 11,900 cfs. The storage need is to secure additional water supply for agriculture and domestic use and for potential future blending with treated effluent to allow use of the effluent for irrigation as a water conservation measure.

Related Environmental Benefits

Fulfilling the project need brings other benefits to fish, wildlife and habitat, as described below for the funding intent.

The funding intent is to secure an Oregon Senate Bill 839 Water Development Project Grant from the Oregon Water Resources Department for funding support. A condition of the grant is that 25% of the new stored water is released for instream flow augmentation purposes. For example, based on an approximate volume of 400 acre-feet in new storage, the flow release would be approximately 100 acre-feet. The timing of the year and the duration of the release is determined in coordination with the Oregon Department of Fish and Wildlife and NOAA; however, assuming a 30-day duration of summer flow release, the flow rate would be approximately 1.7 cubic feet per second (cfs); assuming a 60-day duration of summer flow release, the flow rate would be approximately 0.8 cfs. Actual periods and timing of releases are subject to ODFW and NOAA input. The released water would be into Currant Creek at the dam outlet works, continuing as additional creek flow downstream into Muddy Creek, then to the John Day River. These example added flows to the creek system are substantial increases to otherwise summer low flows with significant benefits to fish, wildlife and habitat.

The OWRD Initial Review report on the storage permit application points out that "The proposed diversion falls within a high priority area for streamflow restoration under the Oregon Plan for Salmon & Watersheds." Project flow releases as described above are consistent with this area designation.

Construction Cost

The cost of the spillway reconstruction raise the crest and store additional water is estimated at $1.0 million to $1.5 million at this stage of the project. This reflects a cost range of roughly $2,500 to $3,700 per acre-foot of additional stored water.

Operation and Maintenance Cost

Operation and maintenance costs are anticipated to decrease since periodic work to reduce erosion potential in the existing spillway condition will decrease after the project is completed. The existing condition requires additional operation and maintenance effort including reservoir monitoring, outlet works manipulations and periodic placement of riprap in the existing spillway channel.

Cost of Next Best Alternative

The next best alternative that achieves a raised spillway crest and additional water supply is estimated to cost approximately $2.5 to $3.2 million. This alternative involves construction of a new emergency spillway crest that would discharge into a natural drainage channel on lands owned by the United States. The discharge would be conveyed in the natural drainage back to Currant Creek downstream of the dam. This alternative also involves relocation and/or raising of the Wasco County road that provides ingress and egress to the YL-WFR Camp and facilities areas. This project would increase storage in the reservoir by about 760 acre-feet and would increase the area of inundation on lands owned by the United States, although the area remains relatively small.

Expected Public Benefits

The completed project improves safety for the YL-WFR staff, visitors and infrastructure of YL-WFR Camp downstream of the dam from hazards of dam breach during large runoff and flood events, consistent with the purpose of emergency spillways for dams classed by the OWRD with high-hazard potential.

As stated above under “Related Environmental Benefits”, flow releases are planned from the new reservoir storage into Currant Creek according to provisions of the SB 839 grant funding which require that 25% of new storage is released for instream flow purposes. The released flow would proceed down Currant Creek, into Muddy Creek, then to the John Day River with improved water conditions for fish (including steelhead in Muddy Creek below the falls), wildlife and habitat. Time and duration of flow releases will be determined according to input of the ODFW and NOAA (see “Related Environmental Benefits” above)

Conserving water provides public benefits. In the longer-term, YL-WFR is evaluating a plan to use some of the new stored water to mix with treated sewage effluent, resulting in dilution of the water so it can be used for irrigation. Re-use of effluent waters reduces the need for additional fresh water supply.

BLOCK 16. Describe probable effects on the population in the area, including the social and economic aspects, and the rural lifestyles.

Enhanced dam safety, fish habitat enhancement, increase in desired flows and temperature reduction to both Muddy Creek and John Day River. Social and economic aspects include enhanced preservation of important protected fishery with no negative impacts upon the wild and scenic river corridor. Minimal impact on rural lifestyle is anticipated due to the great distance between the proposed project and neighboring private properties other than those of the applicant. See attached vicinity map exhibit. The applicant anticipates a positive effect in all aspects. The Federal Land effected is landlocked by private lands and only accessible by crossing private land presently owned by the applicant.

BLOCK 17. Describe likely environmental effects that the proposed project will have on: (a) air quality; (b) visual impact; (c) surface and ground water quality and quantity; (d) the control or structural change on any stream or other body of water; (e) existing noise levels; and (f) the surface of the land, including vegetation, permafrost, soil, and soil stability.

(a) Air quality: No measurable impact.

(b) Visual impact: There will be no appreciable visual impact other than a slightly larger reservoir surface area.

(c) Surface and ground water quality and quantity: Surface water quality could be improved by increased flows and lower stream temperatures. A slight increase to groundwater recharge could also result from the expansion of the reservoir. Otherwise, no impacts to surface water or groundwater are anticipated.

(d) The control or structural change on any stream or other body of water: No controls or structural changes are anticipated from the project activities.

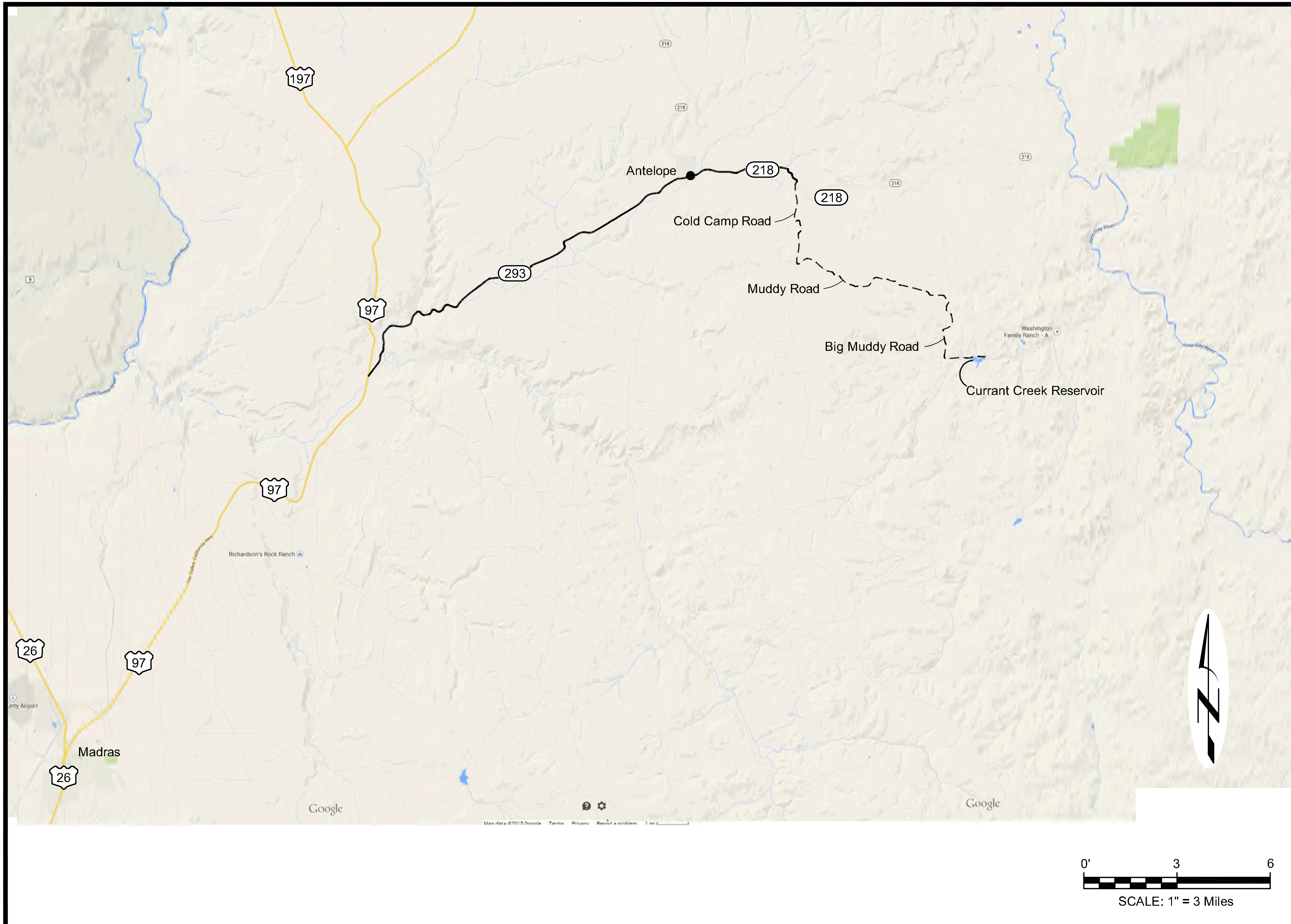
(e) Existing noise levels: Noise levels will only be affected during the construction periods. The dam and spillway modifications will not have any mechanical noise emissions once completed.

(f) The surface of the land, including vegetation, permafrost, soil and soil stability: The dam modifications will improve dam safety. The emergency spillway modifications will reduce scour potential of the spillway channel and improve stability within the spillway.

BLOCK 18. Describe the probable effects that the proposed project will have on (a) populations of fish, plant life, wildlife, and marine life, including threatened and endangered species; and (b) marine mammals, including hunting, capturing, collecting, or killing these animals.

Steelhead migrate up Muddy Creek from the mouth at the John Day River, upstream to a falls on the creek. ODFW and NOAA have strong interest in these fish and the quality of their habitat in Muddy Creek. Under the provisions of a successful SB 839 grant, 25% of the new stored water would be released from the reservoir to Currant Creek for instream flow augmentation during critical periods determined by the ODFW and NOAA as described above in Block 15, under

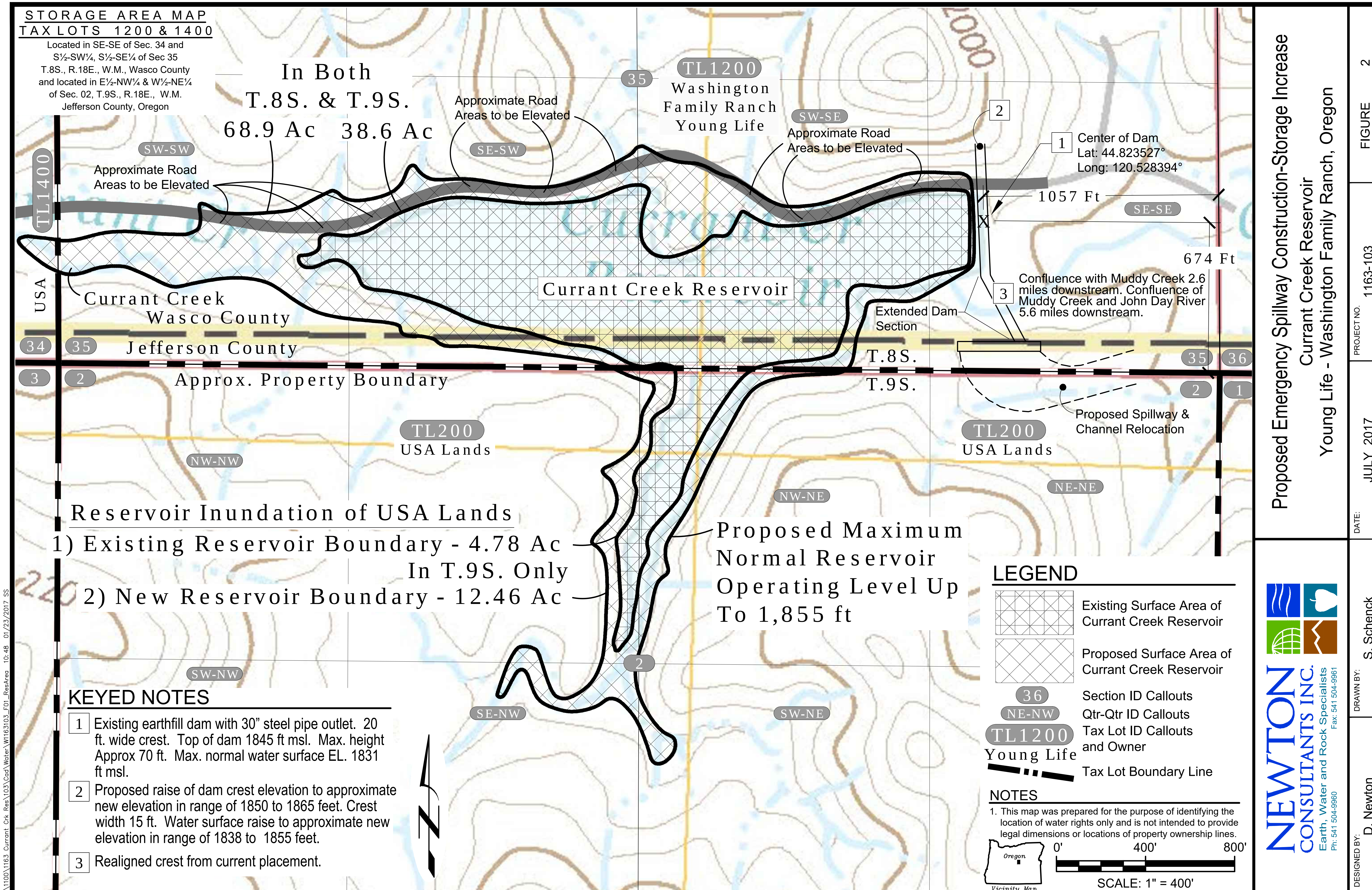
"Related Environmental Benefits". These instream flows would proceed from the reservoir, down Currant Creek, then through Muddy Creek to the John Day River. The added instream flow rates would vary according to the time period over which the releases are scheduled by the ODFW and NOAA. Examples, as stated in Block 15 above, based on 400 acre-feet of new storage and a release volume of 25% of new storage amounts to 100 acre-feet of water, resulting in flow releases of 1.7 cfs and 0.8 cfs, respectively for 30-day and 60-day flow release periods. These are significant contributions to creek flows, which are normally relatively low during summer low-flow conditions. It is anticipated that the timing and duration of flow releases would relate to timing of steelhead presence in Muddy Creek



Vicinity Map
Currant Creek Reservoir
Washington Family Ranch, Oregon

NEWTON CONSULTANTS INC.
Earth, Water and Rock Specialists
Ph: 541 504-9960 Fax: 541 504-9961

FIGURE	1
PROJECT NO.	1163-101
DATE:	FEB 2015
DRAWN BY:	S. Schenck
DESIGNED BY:	J. Newton


STORAGE AREA MAP
TAX LOTS 1200 & 1400
Located in SE-SE of Sec. 34 and S½-SW¼, S½-SE¼ of Sec 35 T.8S., R.18E., W.M., Wasco County and located in E½-NW¼ & W½-NE¼ of Sec. 02, T.9S., R.18E., W.M. Jefferson County, Oregon
In Both T.8S. & T.9S.
68.9 Ac
38.6 Ac
Approximate Road Areas to be Elevated
Approximate Road Areas to be Elevated
Approximate Road Areas to be Elevated
TL1200
Washington Family Ranch Young Life
TL1400
USA
SW-SW
SE-SW
SW-SE
SE-SE
35
Center of Dam
Lat: 44.823527°
Long: 120.528394°
1057 Ft
674 Ft
Confluence with Muddy Creek 2.6 miles downstream. Confluence of Muddy Creek and John Day River 5.6 miles downstream.
Extended Dam Section
Currant Creek Reservoir
Currant Creek
Wasco County
Jefferson County
Approx. Property Boundary
34
35
3
2
35
36
2
1
T.8S.
T.9S.
Proposed Spillway & Channel Relocation
TL200
USA Lands
TL200
USA Lands
NW-NW
NW-NE
NE-NE
SW-NW
SE-NW
SW-NE
Reservoir Inundation of USA Lands
1) Existing Reservoir Boundary - 4.78 Ac
In T.9S. Only
2) New Reservoir Boundary - 12.46 Ac
Proposed Maximum Normal Reservoir Operating Level Up To 1,855 ft
KEYED NOTES
1 Existing earthfill dam with 30" steel pipe outlet. 20 ft. wide crest. Top of dam 1845 ft msl. Max. height Approx 70 ft. Max. normal water surface EL. 1831 ft msl.
2 Proposed raise of dam crest elevation to approximate new elevation in range of 1850 to 1865 feet. Crest width 15 ft. Water surface raise to approximate new elevation in range of 1838 to 1855 feet.
3 Realigned crest from current placement.
LEGEND
Existing Surface Area of Currant Creek Reservoir
Proposed Surface Area of Currant Creek Reservoir
36 Section ID Callouts
NE-NW Qtr-Qtr ID Callouts
TL1200 Young Life Tax Lot ID Callouts and Owner
Tax Lot Boundary Line
NOTES
1. This map was prepared for the purpose of identifying the location of water rights only and is not intended to provide legal dimensions or locations of property ownership lines.
Oregon
Vicinity Map
0' 400' 800'
SCALE: 1" = 400'
Proposed Emergency Spillway Construction-Storage Increase
Currant Creek Reservoir
Young Life - Washington Family Ranch, Oregon
FIGURE 2
PROJECT NO. 1163-103
DATE: JULY 2017
DRAWN BY: S. Schenck
DESIGNED BY: D. Newton
NEWTON CONSULTANTS INC.
Earth, Water and Rock Specialists
Ph: 541 504-9960
Fax: 541 504-9961

CURRANT CREEK DAM

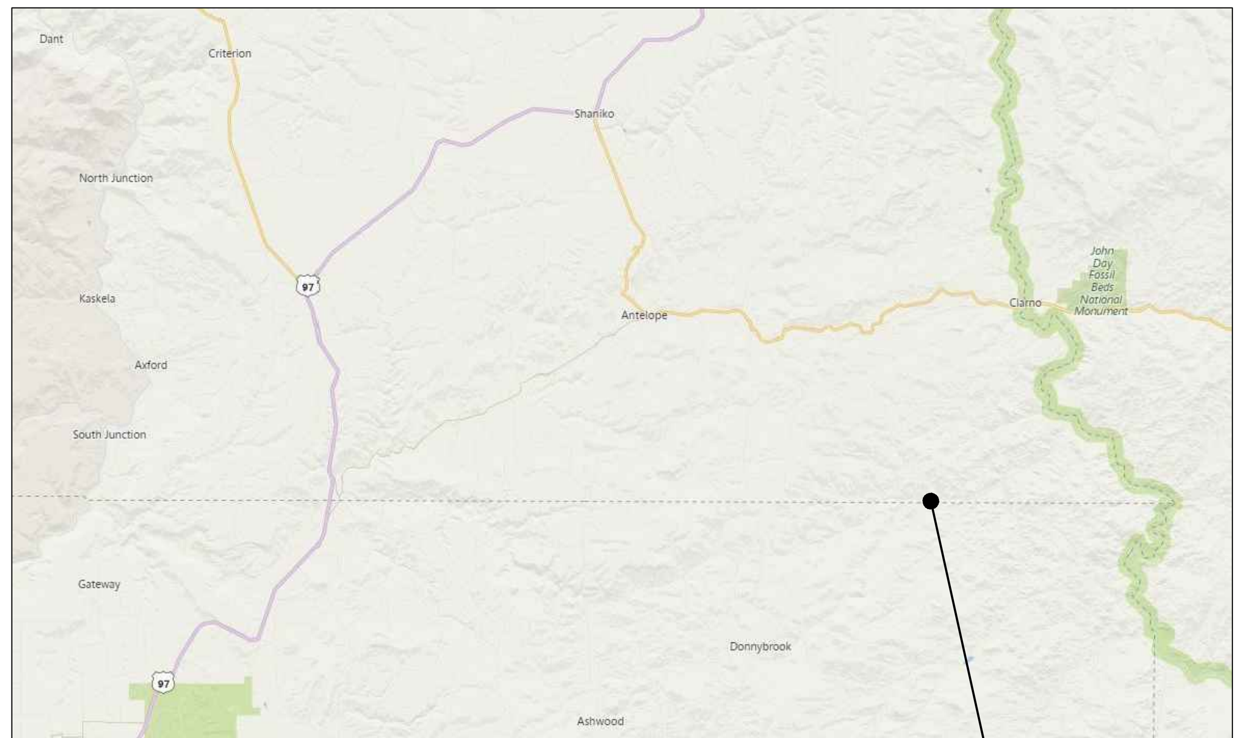
TOPOGRAPHIC SURVEY

WASCO COUNTY AND JEFFERSON COUNTY, OREGON

TAX LOT 1200, MAP 08S18E

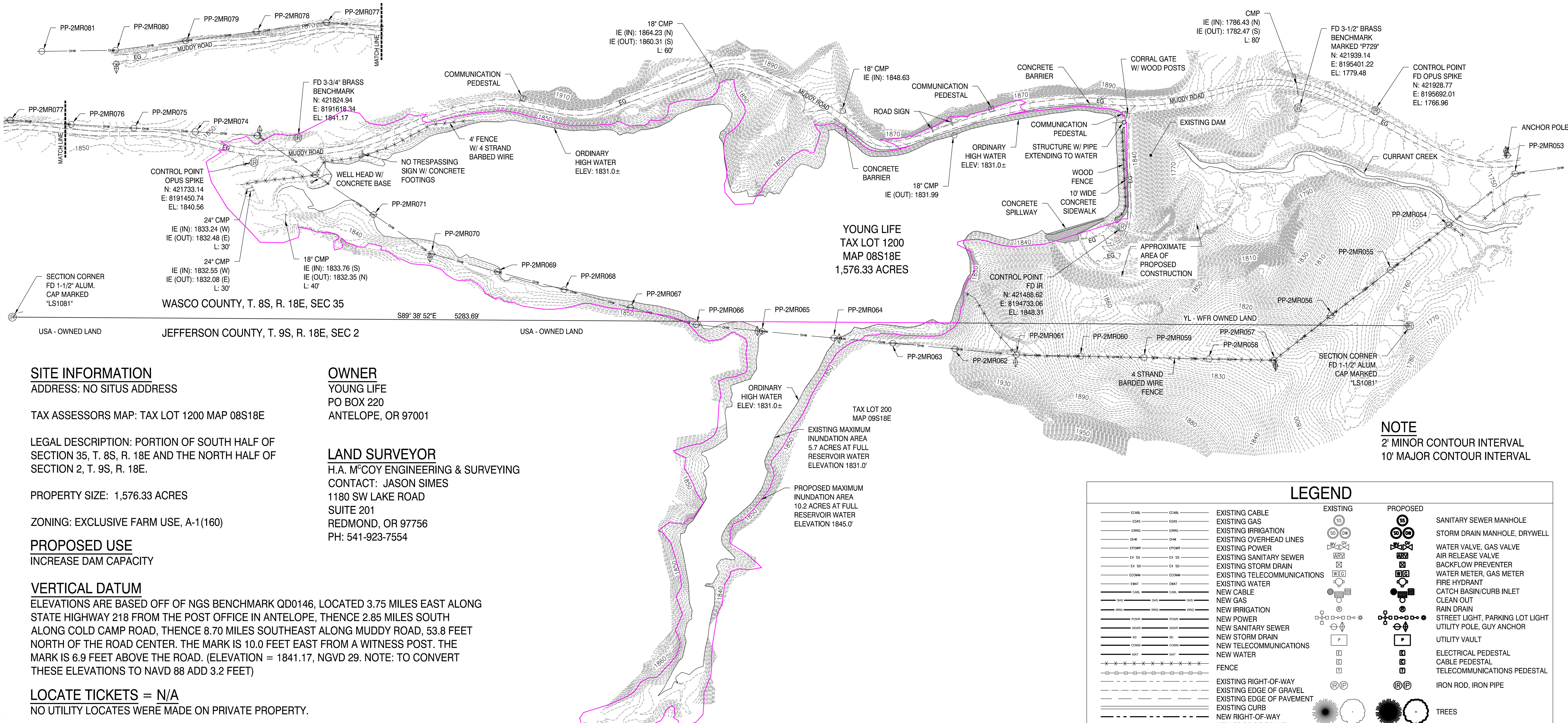
FEBRUARY, 2019

SCALE: 1" = 200'
FOR 34"x22" SHEETS
0 100' 200' 400'



VICINITY MAP

NTS



SITE INFORMATION

ADDRESS: NO SITUS ADDRESS

TAX ASSESSORS MAP: TAX LOT 1200 MAP 08S18E

LEGAL DESCRIPTION: PORTION OF SOUTH HALF OF SECTION 35, T. 8S, R. 18E AND THE NORTH HALF OF SECTION 2, T. 9S, R. 18E.

PROPERTY SIZE: 1,576.33 ACRES

ZONING: EXCLUSIVE FARM USE, A-1(160)

PROPOSED USE

INCREASE DAM CAPACITY

VERTICAL DATUM

ELEVATIONS ARE BASED OFF OF NGS BENCHMARK QD0146, LOCATED 3.75 MILES EAST ALONG STATE HIGHWAY 218 FROM THE POST OFFICE IN ANTELOPE, THENCE 2.85 MILES SOUTH ALONG COLD CAMP ROAD, THENCE 8.70 MILES SOUTHEAST ALONG MUDDY ROAD, 53.8 FEET NORTH OF THE ROAD CENTER. THE MARK IS 10.0 FEET EAST FROM A WITNESS POST. THE MARK IS 6.9 FEET ABOVE THE ROAD. (ELEVATION = 1841.17, NGVD 29. NOTE: TO CONVERT THESE ELEVATIONS TO NAVD 88 ADD 3.2 FEET)

LOCATE TICKETS = N/A

NO UTILITY LOCATES WERE MADE ON PRIVATE PROPERTY.

OWNER

YOUNG LIFE
PO BOX 220
ANTELOPE, OR 97001

LAND SURVEYOR

H.A. McCOY ENGINEERING & SURVEYING
CONTACT: JASON SIMES
1180 SW LAKE ROAD
SUITE 201
REDMOND, OR 97756
PH: 541-923-7554

NOTE

2' MINOR CONTOUR INTERVAL
10' MAJOR CONTOUR INTERVAL

LEGEND

EXISTING / NEW LINES		EXISTING	PROPOSED	
EXISTING CABLE				SANITARY SEWER MANHOLE
EXISTING GAS				STORM DRAIN MANHOLE, DRYWELL
EXISTING IRRIGATION				WATER VALVE, GAS VALVE
EXISTING OVERHEAD LINES				AIR RELEASE VALVE
EXISTING POWER				BACKFLOW PREVENTER
EXISTING SANITARY SEWER				WATER METER, GAS METER
EXISTING STORM DRAIN				FIRE HYDRANT
EXISTING TELECOMMUNICATIONS				CATCH BASIN/CURB INLET
EXISTING WATER				CLEAN OUT
NEW CABLE				RAIN DRAIN
NEW GAS				STREET LIGHT, PARKING LOT LIGHT
NEW IRRIGATION				UTILITY POLE, GUY ANCHOR
NEW POWER				UTILITY VAULT
NEW SANITARY SEWER				ELECTRICAL PEDESTAL
NEW STORM DRAIN				CABLE PEDESTAL
NEW TELECOMMUNICATIONS				TELECOMMUNICATIONS PEDESTAL
NEW WATER				IRON ROD, IRON PIPE
FENCE				TREES
EXISTING RIGHT-OF-WAY				SIGN
EXISTING EDGE OF GRAVEL				MAILBOX
EXISTING EDGE OF PAVEMENT				
EXISTING CURB				
NEW RIGHT-OF-WAY				
NEW EDGE OF GRAVEL				
NEW EDGE OF PAVEMENT				
NEW CURB				

EXISTING CONDITIONS

REGISTERED PROFESSIONAL LAND SURVEYOR
PRELIMINARY
OREGON NOVEMBER 10, 2009 JASON L. SIMES 83256PLS
EXPIRES 6/30/20

DRAWING STATUS:	DATE:	No.	REVISION:	DATE:
TOPO. SURVEY	10/10/17			
ADD'L TOPO	11/08/17			
☒ APPLICATION	02/25/19			

H.A. McCOY ENGINEERING & SURVEYING, LLC
1180 SW Lake Road, Suite 201 Redmond, OR 97756
Hayes McCoy PE, PLS, CWRE
541-923-7554 hayes@ham-engr.com

PROJECT: CURRANT CREEK DAM
PROJECT LOCATION: WASCO COUNTY, OREGON
CLIENT: YOUNG LIFE

SHEET TITLE: TOPOGRAPHIC SURVEY & EXISTING CONDITIONS

JOB NO. 17-070
DRAWN BY: BRG
DRAWING: FIGURE 3

FILE: \\freenas\NAS\HAM Engr\17-070 Current Creek Dam\Engineering\190225-17070-Surface Areas.dwg 2/25/2019 3:21 PM - lon

THE PROPOSED NEW MAXIMUM RESERVOIR WATER ELEVATION WILL INUNDATE APPROXIMATELY 10.2 ACRES OF U.S. LAND. A GRANT IS NECESSARY FROM BLM TO ALLOW THIS. THE ACCOMPANYING SF-299 APPLICATION WAS SUBMITTED TO BLM ACCORDING TO THIS REQUIREMENT UNDER 43 CFR PART 2800.

43 CFR Part 2800 - RIGHTS-OF-WAY UNDER THE FEDERAL LAND POLICY AND MANAGEMENT ACT

CFR

Authority:
43 U.S.C. 1733, 1740, 1763, and 1764.

Source:
70 FR 21058, Apr. 22, 2005, unless otherwise noted.

CFR Toolbox

Law about... Articles from Wex

43 CFR § 2801.2 - What is the objective of BLM's right-of-way program?

CFR

§ 2801.2 What is the objective of BLM's right-of-way program?

It is BLM's objective to grant rights-of-way under the regulations in this part to any qualified individual, business, or government entity and to direct and control the use of rights-of-way on public lands in a manner that:

(a) Protects the natural resources associated with public lands and adjacent lands, whether private or administered by a government entity;

(b) Prevents unnecessary or undue degradation to public lands;

(c) Promotes the use of rights-of-way in common considering engineering and technological compatibility, national security, and land use plans; and

(d) Coordinates, to the fullest extent possible, all BLM actions under the regulations in this part with state and local governments, interested individuals, and appropriate quasi-public entities.

CFR Toolbox

Law about... Articles from Wex

43 CFR § 2801.9 - When do I need a grant?

CFR Table of Popular Names

§ 2801.9 When do I need a grant?

(a) You must have a grant under this part when you plan to use public lands for systems or facilities over, under, on, or through public lands. These include, but are not limited to:

(1) Reservoirs, canals, ditches, flumes, laterals, pipelines, tunnels, and other systems which impound, store, transport, or distribute water;

(2) Pipelines and other systems for transporting or distributing liquids and gases, other than water and other than oil, natural gas, synthetic liquid or gaseous fuels, or any refined products from them, or for storage and terminal facilities used in connection with them;

(3) Pipelines, slurry and emulsion systems, and conveyor belts for transporting and distributing solid materials and facilities for storing such materials in connection with them;

(4) Systems for generating, transmitting, and distributing electricity, including solar and wind energy development facilities and associated short-term actions, such as site and geotechnical testing for solar and wind energy projects;

(5) Systems for transmitting or receiving electronic signals and other means of communication;

(6) Transportation systems, such as roads, trails, highways, railroads, canals, tunnels, tramways, airways, and livestock driveways; and

(7) Such other necessary transportation or other systems or facilities, including any temporary or short-term surface disturbing activities associated with approved systems or facilities, which are in the public interest and which require rights-of-way.

(b) If you apply for a right-of-way grant for generating, transmitting, and distributing electricity, you must also comply with the applicable requirements of the Federal Energy Regulatory Commission under the Federal Power Act of 1935, 16 U.S.C. 791a *et seq.*, and 18 CFR chapter I.

(c) See part 2880 of this chapter for information about authorizations BLM issues under the Mineral Leasing Act for transporting oil and gas resources.

(d) All systems, facilities, and related activities for solar and wind energy projects are specifically authorized as follows:

(1) Energy site-specific testing activities, including those with individual meteorological towers and instrumentation facilities, are authorized with a short-term right-of-way grant issued for 3 years or less;

(2) Energy project-area testing activities are authorized with a short-term right-of-way grant for an initial term of 3 years or less with the option to renew for one additional 3-year period under § 2805.14(h) when the renewal application is accompanied by an energy development application;

(3) Solar and wind energy development facilities located outside designated leasing areas, and those facilities located inside designated leasing areas under § 2809.17(d)(2), are authorized with a right-of-way grant issued for up to 30 years (plus the initial partial year of issuance). An application for renewal of the grant may be submitted under § 2805.14(g);

(4) Solar and wind energy development facilities located inside designated leasing areas are authorized with a solar or wind energy development lease when issued competitively under subpart 2809. The term is fixed for 30 years (plus the initial partial year of issuance). An application for renewal of the lease may be submitted under § 2805.14(g); and

(5) Other associated actions not specifically included in § 2801.9(d)(1) through (4), such as geotechnical testing and other temporary land disturbing activities, are authorized with a short-term right-of-way grant issued for 3 years or less.

[70 FR 21058, Apr. 22, 2005, as amended at 81 FR 92207, Dec. 19, 2016]

In order to raise the dam elevation and increase water storage capacity at the Currant Creek Reservoir, it is necessary to raise the road surface elevation of Muddy Road and avoid inundation. We have determined a cross section that the road must be constructed to according to Wasco County's Development Code and Standard Specification for Construction. In addition, we are coordinating with Wasco County Public Works in order to ensure that the design and construction of the road meets all applicable standards. The attached email, from Arthur Smith, Wasco County Public Works Director, demonstrates acknowledgement and coordination between our project and Wasco County.

Muddy Road Raise-Washington Family Ranch

Arthur Smith <arthurs@co.wasco.or.us> Thu, May 28, 2020 at 8:21 AM
To: Sean Williams <sean@ham-engr.com>
Cc: Jeff McCall <jeffm@co.wasco.or.us>, Dave Newton <dnewton@newtonconsultants.com>, Gabe Williams <gabe@rsiengr.com>, Benn Eilers <benn@rsiengr.com>

Sean,

While this section of the Muddy Road is a public road of local access, per ORS 368.031, Wasco County does exercise jurisdiction over this road. As such, all work performed on this local access road shall comply with the specifications and standards of Wasco County and the work will be subject to review by the Public Works Department. Any questions, please feel free to contact me. Thank you.

Arthur

On Thu, May 28, 2020 at 7:26 AM Sean Williams <sean@ham-engr.com> wrote:

> Arthur,
>
> We spoke about raising Muddy Road at the Currant Creek Dam for Young Life back in February. We are submitting our grant application today, and could use some documentation that Wasco County will be the reviewing authority on the road raise. The plans are not yet complete, this is just a progress set for the grant application, however I've attached the latest road design for reference. I'm not asking for any sort of approval or endorsement of the project, simply a statement that the road will need to be constructed to Wasco County Standards and will be reviewed by Wasco County Public works.
>
> Please let me know if you need any more information.
>
> Sincerely,
>
> Sean Williams, EIT
>
> ***H.A. McCoy Engineering & Surveying, LLC***
>
> Office: (541)923-7554
>
> Cell: (541)863-9261

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WASCO COUNTY
Pioneering pathways to prosperity.

In an effort to prevent, slow, and stop the spread of COVID-19 to our citizens, our office will be limiting business to phone, email and online service. If you are not sure how to access services online, or you have a need that requires in-person assistance, please call our office at 541-506-2640 to discuss. Please keep in mind that response time may vary depending on staffing. Thank you for your patience during this time.

Arthur Smith | ***Director***
PUBLIC WORKS

arthurs@co.wasco.or.us | www.co.wasco.or.us
541-506-2645 | Fax 541-506-2641
2705 East 2nd Street | The Dalles, OR 97058

Attachment 3

Documentation of matching funds

3-1 - Match letter from Young Life (2p)

WASHINGTON FAMILY RANCH
younglife CAMP

May 28, 2020

Ms. Rebecca Williams
Grant Program Coordinator
State of Oregon Water Resources Department

Re: Currant Creek Storage, Creek Flow Augmentation and Fish Passage Project

Dear Ms. Williams:

Several years ago Young Life learned about the Oregon Water Resources Department Grants and Loans opportunities by way of Keith Mills, Oregon Water Resources Dam Safety Officer. Keith provides inspection and evaluation of our Currant Creek Dam and Spillway at Young Life's Washington Family Ranch. This letter is in response to this initial introduction and the culmination of many years of study. In this time we have become familiar with the nuances of the Currant and Muddy Creek watershed and ecology, clearly defined our objectives and proposed design, engaged and collaborated with Native American tribes, state, and federal agencies affected by this proposal, and applied for this grant opportunity. This program has opened the door to possibilities that have not been plausible to date.

Young Life is committed to funding this proposed project with a total match of $961,540. These secured matching funds are comprised of $445,130 of in kind contributions and a cash match of $516,410. In an effort to convey our conviction and commitment to this project we are willing to exceed the 25% required match of the overall projected cost as proposed.

This project will provide many public benefits. These benefits which are outlined in our application include, the retention of jobs in both Wasco and Jefferson county, sustenance and increased economic activity at Washington Family Ranch and the local area, innovation and increased efficiency with an appropriate weir design and fish passage structure, improved agricultural production with additional water provision for both crops and livestock, improved surface water quality and ecosystem resiliency to climate change, decreased water temperature, ecologic enhancements with the addition of a fish passage system to further increase the migration of endangered steelhead and sensitive redband trout, and the opportunity to increase recreation , tourism and education for visiting guests, while promoting state and local priorities.

In the event Young Life's Washington Family Ranch is awarded this grant the following enhancements will take place:

- The Currant Creek Reservoir spillway will be reconstructed and associated dam raised to improve the longevity and safety of this structure. With this increase of storage to the reservoir, local operational out of stream needs will be enhanced, further sustaining jobs and the local operational uses.
- This improvement to the spillway and dam also increases water availability to be applied to in stream flow protection of fish and ecological systems within the John Day River watershed.

Washington Family Ranch 1 Muddy Rd, P.O. Box 220 Antelope, OR 97001 tel. 541 489 3100 fax 541 306 6639 wfr@wfr.younglife.org washingtonfamilyranch.younglife.org

- Fish migration will be improved with the implementation of passage beyond the current Muddy falls obstacle. This will open several more miles of the Muddy Creek drainage that is currently inaccessible.

We are thankful to be considered for this unique and valuable grant opportunity that could improve the ecology and community of this unique location for the greater John Day River watershed in Central Oregon.

Respectfully,

David Briggs
Vice President of Financial Service
Young Life

Washington Family Ranch 1 Muddy Rd, P.O. Box 220 Antelope, OR 97001 tel. 541 489 3100 fax 541 306 6639 wfr@wfr.younglife.org washingtonfamilyranch.younglife.org

Attachment 4

Project feasibility documentation

4-1 – Preliminary Feasibility Memorandum (16p)
4-2 – Site Suitability – Geotechnical Report (89p)
4-3 – Hydrology and Inflow Design Storage Capacity and Breach Analysis (112p)

NEWTON
CONSULTANTS INC.
Earth, Water and Rock Specialists

P.O. Box 1728, 1937 N Business 97
Redmond, Oregon 97756
Phone: (541) 504-9960 FAX: (541) 504-9961

REGISTERED PROFESSIONAL ENGINEER 9975 OREGON OCTOBER 6, 1978 DAVID J. NEWTON
EXPIRES: 12/31/2016

Memorandum

REGISTERED PROFESSIONAL OREGON JAMES B. NEWTON G-2122 GEOLOGIST

To:	Bill Palmaymesa Water Quality/Water Resources Manager Washington Family Ranch 1 Muddy Road, P.O. Box 220 Antelope, Oregon 97001	**Date:**	February 13, 2015
		From:	James B. Newton, R.G., E.I.T., C.W.R.E. David J. Newton, P.E., C.E.G., C.W.R.E.
		Project Name:	Currant Creek Reservoir Washington Family Ranch
Subject:	Storage Increase Feasibility; Currant Creek Reservoir	**Project No.:**	1163-101

Dear Bill:

This memorandum summarizes an evaluation by Newton Consultants, Inc. (Newton) of alternatives for increasing storage capacity in the existing Current Creek Reservoir (Reservoir). The Reservoir is located in Sections 35 and 2, Townships 8 and 9 South, Range 18 East, W.M., Wasco County, Oregon. The site location is shown on Figure 1.

PURPOSE

The purpose of the evaluation is to assist the Washington Family Ranch (Ranch) in making decisions relative to feasibility of increasing water supply through additional storage capacity in the Reservoir. Spring runoff from the Reservoir watershed suggests that additional water is available that could be stored for Ranch uses, rather than passing it through the Reservoir. The existing emergency spillway is eroded, although it is currently operable with performance observations and related maintenance, if necessary. It may be possible to construct a new emergency spillway at a new location with higher elevation such that erosion maintenance could be reduced and additional storage capacity could also be achieved in the Reservoir. The evaluation is intended to provide the Ranch with preliminary concepts and cost estimates for relocation of the emergency spillway in response to erosion issues with the existing spillway and raising the existing earth fill dam to increase storage capacity. This information is intended for Ranch use in deciding whether or not to proceed with these modifications.

BACKGROUND

The Reservoir is currently authorized to store 1020 acre-feet of water for multi-use purposes use under certificate 89219 issued by the Oregon Water Resources Department (OWRD). Use of water from the Reservoir under additional water rights is to meet the multi-use needs of the Ranch, including irrigation, recreation and potable water needs. Additional storage in the Reservoir would require an additional storage permit from the Oregon Water Resources Department (OWRD).

SITE RECONNAISSANCE

Currant Creek Dam

Newton conducted a reconnaissance of the Reservoir on December 18, 2014 with Bill Palmaymesa of the Ranch, and survey staff from Dejarnatt Surveys, Inc. (DSI) to observe the existing dam and emergency spillway, terrain conditions in potential new spillway locations, and to obtain spot elevations at locations on the top of the existing dam and in the existing emergency spillway. Spot elevations were also determined in the two areas considered by the Ranch for construction of a new emergency spillway. The spot elevations were intended to provide preliminary information on the differences in elevation between potential new spillway inlets and the top of the existing dam. This provides a preliminary idea of the range in heights to which the dam could be raised to store additional water.

Observations during the site reconnaissance indicate that the existing dam is in relative good condition. The existing emergency spillway has been eroded downstream of the entrance and control section. Erosion has occurred in the steeper part of the spillway channel by water-cutting into weathered and relatively soft tuffaceous volcanic rocks. Rock had been placed in the erosion area by the Ranch and by others. The spillway is functional; however, future continued maintenance is likely to help manage erosion and deterioration of the spillway channel. Other site reconnaissance observations are summarized below.

- The observed slope angles on the existing dam are up to approximately 3 to 1 (horizontal to vertical measurement units) on the upstream slope (reservoir side) and up to approximately 2 to 1 on the downstream slope. These angles are consistent with the original design of the dam and they are consistent with slope angles generally used in practice for earth fill dams. These slope angles are generally supported by the OWRD for earthen fill dams, unless steeper slopes are justified by slope stability analyses or alternate construction materials are proposed.

- The upstream slope of the dam is blanketed with riprap from the toe to the top of the slope. Riprap consists of durable basalt and ranges in size from approximately 4 inches to about 2 feet, more or less. No slough areas were observed.

- The downstream slope consists of earth fill materials used to construct the dam. The slope is moderately well-covered with grasses. No sloughing areas were noted on the slope.

- The top of the dam is relatively flat, capped with 3/4-inch minus crushed rock and is trafficable to motor vehicles. The top width of the dam is approximately 20 feet. The spillway channel is approximately 200 feet wide at the inlet. The channel conveys flow from the spillway to its confluence with the channel of Currant Creek. The spillway channel was constructed with a layer of reasonably well-graded, durable, andesite riprap and broken up concrete blocks. Sizes of rock generally range from approximately 4 inches up to 18 inches, with some rocks larger. The thickness of riprap appears to be up to about 4 feet. Although no head-cut was observed in the spillway channel, the channel was incised to bedrock in areas and discussions with Mr. Palmaymesa indicate that Ranch must periodically augment the riprap in the spillway with new riprap to maintain the integrity of the spillway channel.

- Inlet works were submerged under water at both the inlet and were not observed at the time of the site reconnaissance on December 18.

- The condition and integrity of the existing dam for potential modifications are good considering reconnaissance observations and known construction procedures, construction inspections and compaction testing of the dam embankment during its construction under the oversight of David Newton, P.E., C.E.G.

FEASIBILITY CONSIDERATIONS

Availability of Water for Additional Storage

Although spring and early summer runoff through the Reservoir suggests that water is available for additional storage, regulatory restrictions can exist which limit the amounts that are actually available for storage. Restrictions can include existing senior water rights downstream from the Reservoir for instream flows, scenic waterway flows and other purposes.

The OWRD Water Availability Reporting System (WARS) was reviewed to evaluate availability of water for storage above the confluence of Muddy Creek with the John Day River (the nearest applicable gaged watershed for water availability). This availability analysis is based on existing senior water rights. Based on WARS water is available for storage based on 50 percent exceedance criteria (water is generally available 5 out of 10 years) in the stated amounts during the storage period of November 1 to June 30 of each year. Findings relative to water availability are presented in the following Table 1.

Month	50% Exceedance Data - Muddy Creek - Net Water Available (cfs)	50% Exceedance Data - Currant Creek (35.1 %) - Net Water Available (cfs)	Available Water Volume - Currant Creek (acre-ft)
Jan	2.6	0.91	52.02
Feb	7.0	2.46	140.05
Mar	10.7	3.76	214.07
Apr	4.0	1.40	80.03
May	1.3	0.46	26.01
Jun	0.2	0.07	4.00
Jul	-	-	-
Aug	-	-	-
Sep	-	-	-
Oct	-	-	-
Nov	1.3	0.46	26.01
Dec	2.1	0.74	42.01
Total	29.2	10.2	584.20

Table 1. Table 1 above shows the available 50% exceedance for Muddy Creek at the confluence with the John Day River. Currant Creek is approximately 35.1 % of the total watershed area of the Muddy Creek watershed. Assuming a direct relationship that the Currant Creek watershed yields approximately 35.1% of available water for storage to Currant Creek, the annual 50% exceedance available for storage is estimated at 584.2 acre-feet.

Dam Hazard Class – Spillway Capacity Requirement

Discussion with OWRD Dam Safety Division engineer Keith Mills confirmed that the Reservoir dam is presently classified by the OWRD as a high hazard dam. Any modifications to the dam and spillway would include provisions that require the spillway to pass the flow generated by the General Storm Probable Maximum Flood (PMF) event.

The design flow capacity of the existing emergency spillway was checked by Review of the report *Geotechnical and Hydrologic Analyses and Technical Specifications for Currant Creek Reservoir, March 1983* (Report), prepared by David Newton of Newton Consultants (formerly of Century West Engineering Corporation) for the Reservoir. The reported peak flow, or General Storm PMF flow (PMF) for which the existing emergency spillway is designed is 22,400 cubic feet per second (cfs). Discussion with Mr. Mills confirmed that emergency spillway modifications must include provisions that provide capacity to pass the minimum flow rate of 22,400 cfs.

Condition and Integrity of the Existing Dam

As noted in the site reconnaissance section of this memorandum, the observed condition of the existing dam is relatively good. The integrity of the dam is also good based on direct knowledge of David Newton relative to the construction process, construction observations and compaction testing of the embankment during construction under his oversight. The condition and integrity of the existing dam are important in planning and implementing modifications to raise the structure for increasing reservoir storage.

ALTERNATIVES FOR INCREASING STORAGE CAPACITY

Basis for Evaluation of Alternative Concepts

Alternatives for modifying the emergency spillway and the existing dam are based on the current configuration of Reservoir, existing dam, existing spillway and spot elevations provided by DSI at locations on the existing dam and spillway and at locations in areas for potential new spillway construction, and on U.S. Geological Survey topographic maps.

Baseline Storage Conditions

The existing dam and Reservoir has capacity to operate at a maximum water surface elevation of 1831 feet above mean sea level (msl) based on the above-cited report. The approximate elevation of the top of the dam is 1,845 feet. The top elevation of the existing dam is 14 feet above the lowest part of the emergency spillway crest. With an estimated emergency spillway flow depth of 8.3 feet to pass the estimated general storm PMF flow event, the dam currently has 5.7 feet of freeboard (elevation difference between top of dam and maximum water surface elevation under the estimated general storm PMF flow event). The OWRD Dam Safety Division requires at least 2 feet of freeboard. The elevation of the maximum normal operating water surface in the Reservoir is 1,831 feet. The approximate surface area of the Reservoir when full is 44 acres. The maximum storage capacity of the Reservoir is presently 1,020 acre-feet at a water surface elevation of 1,831 feet msl. The Reservoir storage volume in relation to water surface elevation is shown on the area-capacity curve of Figure 2.

Baseline Elevation Conditions – Two Potential New Spillway Locations

Two potential locations for a new emergency spillway exist in a relatively high east-west trending ridge line south of the existing emergency spillway. The amount of potential storage increase in the Reservoir depends on the required elevation for the inlet of a new emergency spillway with capacity to pass water at a flow rate of at least 22,400 cfs.

Preliminary evaluation of the spot elevation data and channel cross-section requirements indicate that new emergency spillways could be constructed at both potential sites, providing capacity to store significant additional water.

Alternative I

Two basic alternatives were considered for increasing storage capacity in the existing Reservoir. Alternative I is based on a new emergency spillway at a location on the ridge south of the existing spillway. This location was selected during site reconnaissance with Ranch personnel. At this location, the invert elevation of the spillway entrance was determined by fitting the approximated minimum required spillway cross-section into the ridge in a manner that requires minimal excavation. The spillway invert on this basis would be at an approximate elevation of 1,858.5 feet, resulting in a maximum Reservoir water surface area of about 73 acres.

The present maximum reservoir water surface elevation is about 1,831 feet. Therefore Alternative I would raise the maximum water surface elevation by about 27.5 feet. This would increase maximum storage capacity from 1,020 acre-feet at present, to about 2,378 acre-feet. If the present 14-foot elevation difference between the top of the dam and the normal operating water surface in the Reservoir is maintained, the existing dam would need to be raised 27.5 feet to implement Alternative I.

The volume of water annually available for storage is estimated at 584 acre-feet. Storage capacity for Alternative I is much higher than this available annual volume. Accordingly, Alternative I could be implemented by storing annual increments of 584 acre-feet over some period of years until a target maximum storage volume is obtained. This plan is suggested on the assumption that the Ranch continues use of water from that storage volume it now relies on. The annual increments of new storage would be on top of the normal storage volume the Ranch currently depends on. At times, it may be necessary for the Ranch to use some of the added storage volume. Some of the additional storage would be lost to seepage and evaporation. Annual evaporation at 2.5 feet over a 44 acre surface area could be about 110 acre-feet.

Alternative II

Alternative II is based on a new emergency spillway at a location on the ridge to the west of the Alternative I location, at a higher elevation. Fitting a spillway channel cross-section into this area with minimal excavation results in an invert elevation at the entrance that is about 55 feet higher than the present maximum Reservoir water surface elevation. This would put the maximum reservoir water surface elevation at roughly 1,886 feet. Extrapolation of the area-capacity curve on Figure 2, assuming canyon topography remains consistent with the area of the present reservoir, storage capacity could be on the order of 3,800 acre-feet. This suggests that the volume of new storage could be on the order of 2,800 acre-feet, requiring about 7 years or more of incremental storage of 584 acre-feet to fill the reservoir (the actual time required depends on annual evaporation, seepage and consistency in actual water availability for storage considering drought, Ranch demands on storage, etc.). Assuming that the 14-foot elevation difference between maximum water surface under normal operations and top of dam is to be maintained, the dam would need to be raised by 55 feet to implement Alternative II.

PRELIMINARY CONCEPTS – DAM AND EMERGENCY SPILLWAY MODIFICATIONS

Preliminary concepts of dam and emergency spillway modifications were developed according to the baseline information described above. Illustrative concepts that show basic layouts for dam and spillway modifications are shown on Figure 3 for Alternative I and Alternative II.

Both Alternative I and Alternative II require extension of the existing dam. The extended dam length is to the southeast for Alternative I, connecting to the ridge at an elevation that raises the maximum Reservoir water surface elevation by 27.5 feet. Two scenarios for raising and extending the existing dam were considered for Alternative I. The extended dam length is to the southwest for Alternative II, connecting to the ridge at a higher elevation than for Alternative I. This allows for raising the maximum Reservoir water surface elevation by 55 feet. One scenario was considered for raising and extending the existing dam for Alternative II.

Alternative I – Increase in Height of Existing Dam

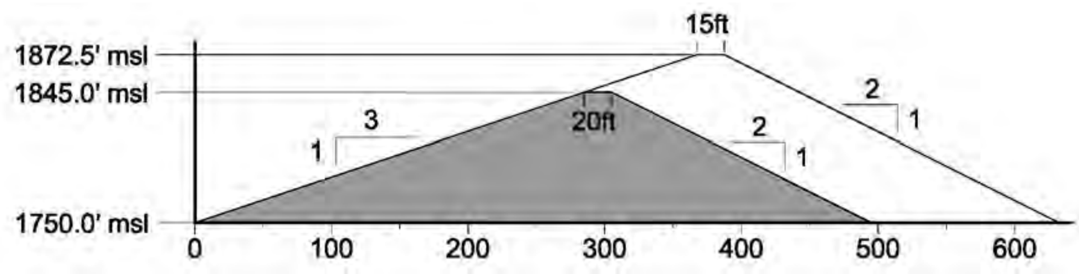
The top elevation of the existing dam is approximately 1,845 feet msl. Raising the maximum Reservoir water surface elevation by 27.5 feet would put the new maximum Reservoir water surface elevation at roughly 1,858.5 feet msl. Accordingly, the existing dam must be raised by 27.5 feet if the 14-foot difference in elevation between the top of the existing dam and existing spillway invert is maintained. This would put the top of the raised dam at an elevation of roughly 1,872.5 feet msl. Water storage capacity would be increased by about 1,358 acre-feet.

Alternative I – Scenarios for Increasing Height of Existing Dam

Two scenarios were considered for raising the existing dam to store an additional water volume of roughly 1,358 acre-feet. Preliminary cross-section concepts were prepared for each scenario. The concepts are intended as a planning basis for estimating volumes of fill material required for each concept and for estimating order of magnitude construction costs. Scenario 1 is based on an engineered earth fill embankment placed against the downstream face of the existing dam and a new fill embankment extending southeast to the potential new emergency spillway site. Scenario 2 is based on a roller-compacted concrete (RCC) dam structure constructed on top of the existing dam.

Scenario 1

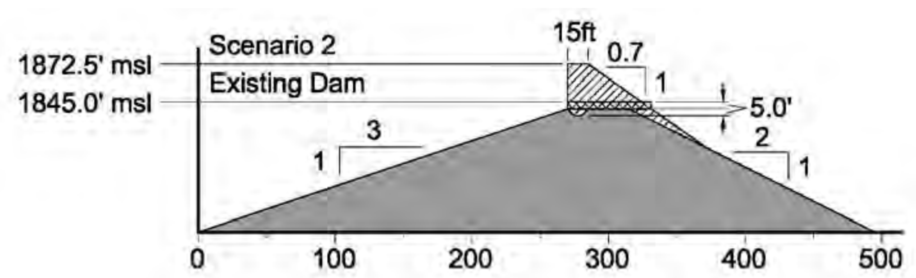
The Scenario 1 concept is based on earth fill embankments to raise the existing dam and to extend the existing dam southeast to a new emergency spillway location. The preliminary cross-section is based on embankments with a 3 horizontal to 1 vertical inclination for the upstream slope and a 2 to 1 inclination for the downstream slope of the new embankment. The top width of the modified dam is contemplated at 15 feet. The Scenario 1 concept is not intended to change the existing section of the dam. Engineered fill would be added to the existing dam. The extended part of the dam to the southeast would be a newly constructed dam. The preliminary cross-section for this concept is shown below.



Scenario 2

The Scenario 2 concept is based on a RCC structure placed on top of the existing dam and a new RCC dam structure extending southeast to the new emergency spillway site. RCC is well-graded (over a range of particle sizes) crushed durable rock generally sized at 3 inches and finer. The crushed rock is blended with about 3 to 5 percent by weight of Portland cement in a batch plant. The mixed RCC is then placed in lifts on the dam construction surface and compacted by heavy smooth-drum rollers to a specified degree of compaction. Care is required in construction to avoid permeable horizontal seams through the dam and to achieve proper compaction. Advantages in RCC use include the reduced volume, time and cost of dam construction.

The Scenario 2 concept is based on a vertical upstream face of the structure and an inclination of 0.7 to 1 for the downstream face of the structure. Foundation improvements on the top of the existing are contemplated for this scenario due to shear stresses applied by the new RCC structure. Accordingly, preliminary consideration was given to removing the top 5 feet of the existing earth fill dam and replacing the fill with an RCC mat. The new RCC dam structure would be supported by this mat, which distributes loads and shear stresses. The specific mat requirement and specific configuration of the RCC dam structure requires stability analysis and engineering evaluations beyond the scope of this feasibility evaluation. The top width of the dam is contemplated at 15 feet. The Scenario 2 concept is illustrated below. The shaded area is the approximate cross section shape of the existing dam.



The above Scenario 2 (RCC dam raise) involves removal of the top 5 feet of earth fill and replacement with an RCC mat, providing an improved foundation for the RCC dam structure. The remaining earth fill section of the existing dam is intended to remain unchanged.

Alternative I – Order of Magnitude Costs – Scenarios 1 and 2

Scenario 1 – Earth Fill

The rough volume of earth fill required to construct the earth fill embankment to raise the existing dam and extend the dam southeast to the potential new emergency spillway site was estimated according to the baseline information described earlier in this memorandum. The total length of embankment construction was estimated at roughly 1,150 feet. Based on the concept illustration above and new dam extension, the volume of earth fill required to construct Scenario 1 is roughly 457,000 cubic yards. Allowing 20 percent for shrinkage (compaction in the embankment), the total required borrow excavation volume is roughly 548,400 cubic yards.

An order of magnitude cost estimate for Scenario 1 is tabulated below:

Scenario 1 - Earth Fill Dam, Raise 27.5 feet				
Material Cost				
	Volume	Unit	Cost/Unit	Extended Cost
Fill	548,400	cy	$6.00	$3,290,400
Appurtenances (e.g. drains, valves, etc.)			8%	$263,232
Subtotal				$3,553,632
Design and Engineering Cost				
Engineering/Construction Oversight/ Survey/Permitting			10%	$329,040
Subtotal				$3,882,672
Contingency			20%	$776,534
TOTAL				**$4,659,206**

Scenario 2 – RCC Dam Structure

The rough volume of RCC required to raise the existing dam and construct the extension of the existing dam southeast to the new potential emergency spillway site was estimated according to the baseline information described earlier in this memorandum. The total length of RCC embankment construction was estimated at roughly 1,150 feet. Based on the Scenario 2 concept illustration above and new dam extension, the volume of RCC required to construct Scenario 1 is roughly 33,900 cubic yards. Roughly 900 cubic yards of Portland cement would be required assuming it is 5 percent by weight of the RCC mix.

An order of magnitude cost estimate for Scenario 2 is shown below:

Scenario 2 - Rock Fill Dam, Raise 27.5 feet				
Material Cost				
	Volume	Unit	Cost/Unit	Extended Cost
Rock Fill	16,000	cy	$12.00	$192,000
Roller Compacted Concrete	17,900	cy	$60.00	$1,074,000
Appurtenances (e.g. drains, valves, etc.)			15%	$189,900
Subtotal				$1,455,900
Design and Engineering Cost				
Engineering/Construction Oversight/ Survey/Permitting			30%	$436,770
Subtotal				$1,892,670
Contingency			30%	$567,801
TOTAL				**$2,460,471**

SUMMARY OF FINDINGS

Based on the estimations of Reservoir storage potential and the current configuration of the Currant Creek Dam, the following findings included herein are summarized below:

- The current condition and integrity of the existing earth fill dam are satisfactory for accommodating potential modifications as described herein. The OWRD Dam Safety Division engineer acknowledges that the overly steep slope of the existing emergency spillway channel has been maintained to a relatively stable condition over many years of water storage in the reservoir. However, any changes to the dam, such as for increase in storage capacity, requires stability analysis for the existing dam and slope conditions under the proposed additional earth fill loading. The spillway does show some signs of erosion that should be addressed; however, overall the spillway and dam appear to operate appropriately in their existing condition.

- Based on a review of the OWRD Water Availability Reporting System, water is available for storage in Current Creek during the storage period of November 1 through June 30 of each year based on 50% exceedance criteria. The annual volume available for storage is estimated at approximately 584 acre-feet.

- The existing dam is classified as a high hazard structure in accordance with OWRD Dam Safety Division criteria. This classification was confirmed by the OWRD Dam Safety Division engineer in communication relative to this feasibility evaluation. Accordingly, the emergency spillway must have capacity to pass the general storm Probable Maximum Flood (PMF) event. The general storm PMF flood flow estimated during original design of the dam and reservoir (above-cited report) is approximately 22,400 cfs. This flow rate was the basis for design of the existing spillway. Reconstruction of the emergency spillway must include capacity to pass the flow rate of 22,400 cfs.

- Two sites for potential emergency spillway reconstruction that would allow additional storage capacity in the existing Reservoir exist on an east-west trending ridge south of the existing spillway. The Alternative I site is located east of the Alternative II site, at a lower elevation than the Alternative II site.

- Reconstruction of the emergency spillway in accordance with Alternative I can raise the maximum Reservoir water surface elevation by about 27.5 feet. Construction is based on the minimum excavation required to construct a channel with capacity to pass 22,400 cfs. This spillway raise would bring the inlet to an approximate elevation of 1,858.5 feet. Accordingly, Alternative I can increase the storage volume of the Reservoir by roughly 1,358 acre-feet.

- Alternative I can provide more reservoir storage capacity than the annual volume of water that is available for storage according to regulatory limitations. However, storage in annual increments of up to 584 acre-feet might be possible to ultimately bring storage in the modified Reservoir up to about 1,358 acre-feet. The time required to achieve this additional storage depends on the annual volume of water use by the Ranch, evaporation, seepage, net volume of water available for storage each year, drought, etc.

- Reconstruction of the emergency spillway in accordance with Alternative II can raise the maximum Reservoir water surface elevation by about 55 feet. Construction is based on the minimum excavation required to construct a channel with capacity to pass 22,400 cfs. This spillway raise would bring the inlet to an approximate elevation of 1,886 feet. Accordingly, Alternative II can increase the storage volume of the Reservoir by roughly 2,900 acre-feet. This rough estimate is based on the existing area-capacity curve and the assumption that it is representative of the large additional water surface area that would result from Alternative II.

- Increasing the height and length of the existing dam to implement Alternative I – Scenario I may require approximately 548,400 cubic yards of earth fill material to construct the earth fill dam components of Scenario 1. The order of magnitude cost estimate for Scenario 1 is $4,659,206.

- Increasing the height and length of the existing dam to implement Alternative I – Scenario 2 may require about 33,900 cubic yards of RCC and about 900 cubic yards of Portland cement to construct the RCC dam components of Scenario 2. The order of magnitude cost estimate for Scenario 2 is $2,460,471.

- An additional review of Scenario 3 that could raise the dam a total height of approximately 55 feet was evaluated. A detailed cost analysis of this Scenario 3, however, was excluded from this memorandum on the basis of the combination of overall earth/RCC material and limited amount of water available for storage (which may prohibit the reservoir from filling in a timeframe of 7 years or more) were not likely reasonable for project feasibility.

ENGINEERING REQUIREMENTS

An overview of general engineering requirements to design the dam and emergency spillway improvements is provided below. This overview is intended as a general guideline for developing a plan to implement the project through the permit process. The construction phase involves different engineering requirements that are defined on the basis of the project design and site conditions that have influence on construction methods and that may require trouble-shooting, field decisions and field modifications to the construction plans and specifications.

1. Review results of completed preliminary evaluations presented in this memorandum with Mr. Palmaymesa. Confirm alternative and scenario for implementation. Develop and implement scope of preliminary engineering and design phase, including requirements 2 through 15 below.

2. Consult with OWRD and ODFW on needs for fish passage in connection with proposed increase in storage capacity.

3. Based on the review with Mr. Palmaymesa from items 1 and 2 above, prepare OWRD storage applications for submittal to OWRD to authorize additional storage in Currant Creek Reservoir.

4. Consult with OWRD Dam Safety Division on coordination and schedule details, and engineering details to be accounted for in developing engineered construction plans and specifications for improvements to increase storage.

5. Conduct topographic and dimensional surveying of the current dam, including the dam profile, spillway and channel reconnection areas. Additional surveying is likely required to document the Reservoir inundation area to refine the Reservoir storage volume, as likely required by the OWRD to comply with permit conditions.

6. Spillway capacity; confirm with OWRD Dam Safety Division, general storm PMF flow rate for spillway capacity considering the peak value of 22,400 cfs.

7. Conduct surface and subsurface investigation of proposed site of new emergency spillway. Investigation is intended to identify types, durability, erosive potential, and excavability of geologic materials that would form the bottom and sidewalls of the new spillway channel.

8. Conduct surface and subsurface investigation of foundation and abutment areas for proposed dam structures. Investigation is intended to identify types of geologic materials, compressibility characteristics, shear strength capacity for bearing pressure determinations, presence of permeable materials or structural features conducive to seepage and internal erosion under the hydraulic pressures of the reservoir.

9. Identify and evaluate potential borrow areas for earth fill materials (Scenario 1) and for rock materials (Scenario 2). Evaluations include consideration of potential volumes of material; consistency of material type and quality; durability, fracture and breakage characteristics of rock; hauling distance to dam site.

10. Obtain representative soil samples of the existing dam embankment and potential borrow areas (soil borrow materials for the Scenario 1 or rock borrow areas for Scenario 2). Arrange and implement testing of selected samples under Scenario 1 for maximum dry density and optimum moisture content; Atterberg Limits (plasticity); grain-size distribution and percentage finer than the No. 200 U.S. Standard Sieve; and direct shear. Arrange and implement testing of selected samples under Scenario 2 for rock quality specifications, including LA Abrasion and Oregon Air Degradation, and rock fracture patterns for rip ability or blasting conditions.

11. Conduct stability analysis of the existing dam and slopes under proposed earth fill or RCC loading conditions and rapid reservoir drawdown. Estimate safety factor for stability of upstream slope of existing dam under additional fill loads and rapid drawdown conditions.

12. Investigate configuration and condition of existing outlet pipe and control valve system. Document type of pipe material, pipe size and operability.

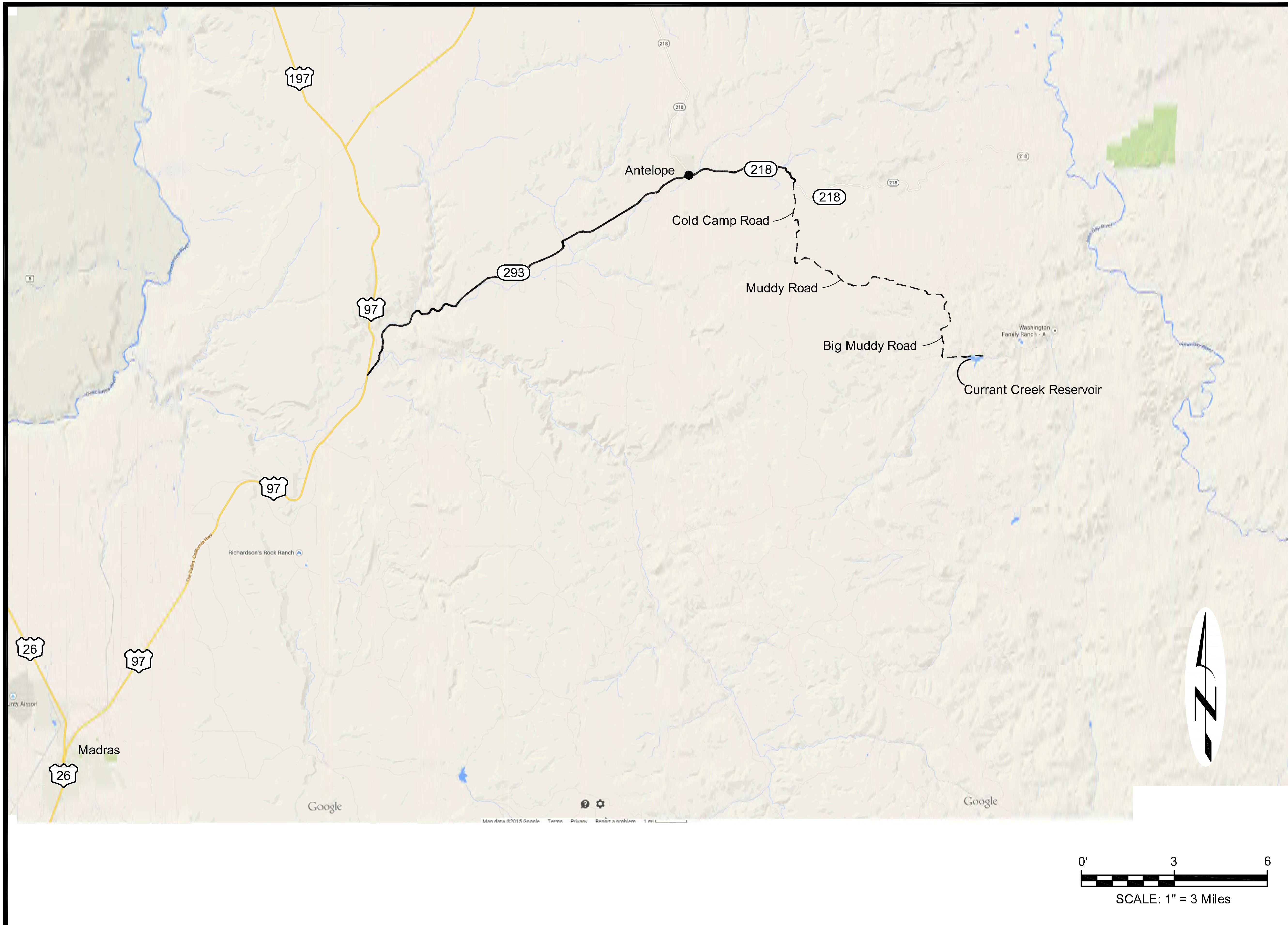
13. Determine modifications to existing outlet pipe and valve control system required by proposed changes in dam configuration to achieve additional storage.

14. Evaluate settlement potential of existing dam embankment under proposed additional earth or RCC loads, and potential impact on existing outlet pipe and control valve system.

15. Prepare geotechnical engineering report establishing basis for design and specifications for dam modifications, design of new emergency spillway, spillway riprap and other potential modifications. Report will include test pit and/or boring logs, soil test results, stability analysis and settlement analysis.

16. Prepare draft engineering construction plans and specifications for review by Mr. Palmaymesa and by OWRD Dam Safety Division.

17. Finalize engineering construction plans and specifications and submit to the OWRD Dam Safety Division.



Vicinity Map
Currant Creek Reservoir
Washington Family Ranch, Oregon

FIGURE 1

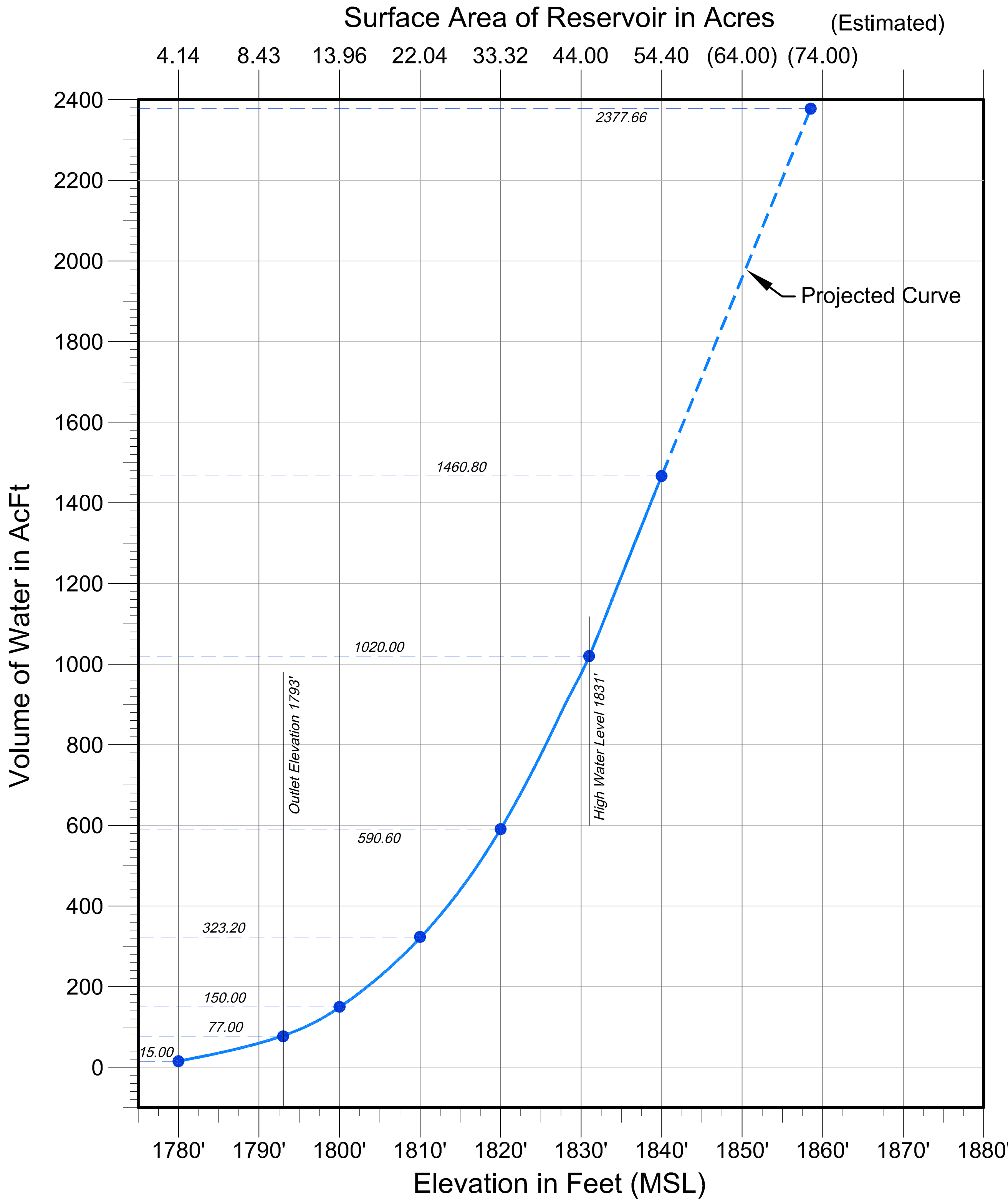
PROJECT NO. 1163-101

DATE: FEB 2015

DRAWN BY: S. Schenck

DESIGNED BY: J. Newton

NEWTON CONSULTANTS INC.
Earth, Water and Rock Specialists
Ph: 541 504-9960 Fax: 541 504-9961



Reference: Geotechnical & Hydrologic Analyses and
Technical Specifications for Currant Creek Reservoir
Figure 3, Area-Capacity Curves
Dated: March, 1982
By: Century West Engineering Corporaation

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Earth, Water and Rock Specialists
Ph: 541 504-9960 Fax: 541 504-9961

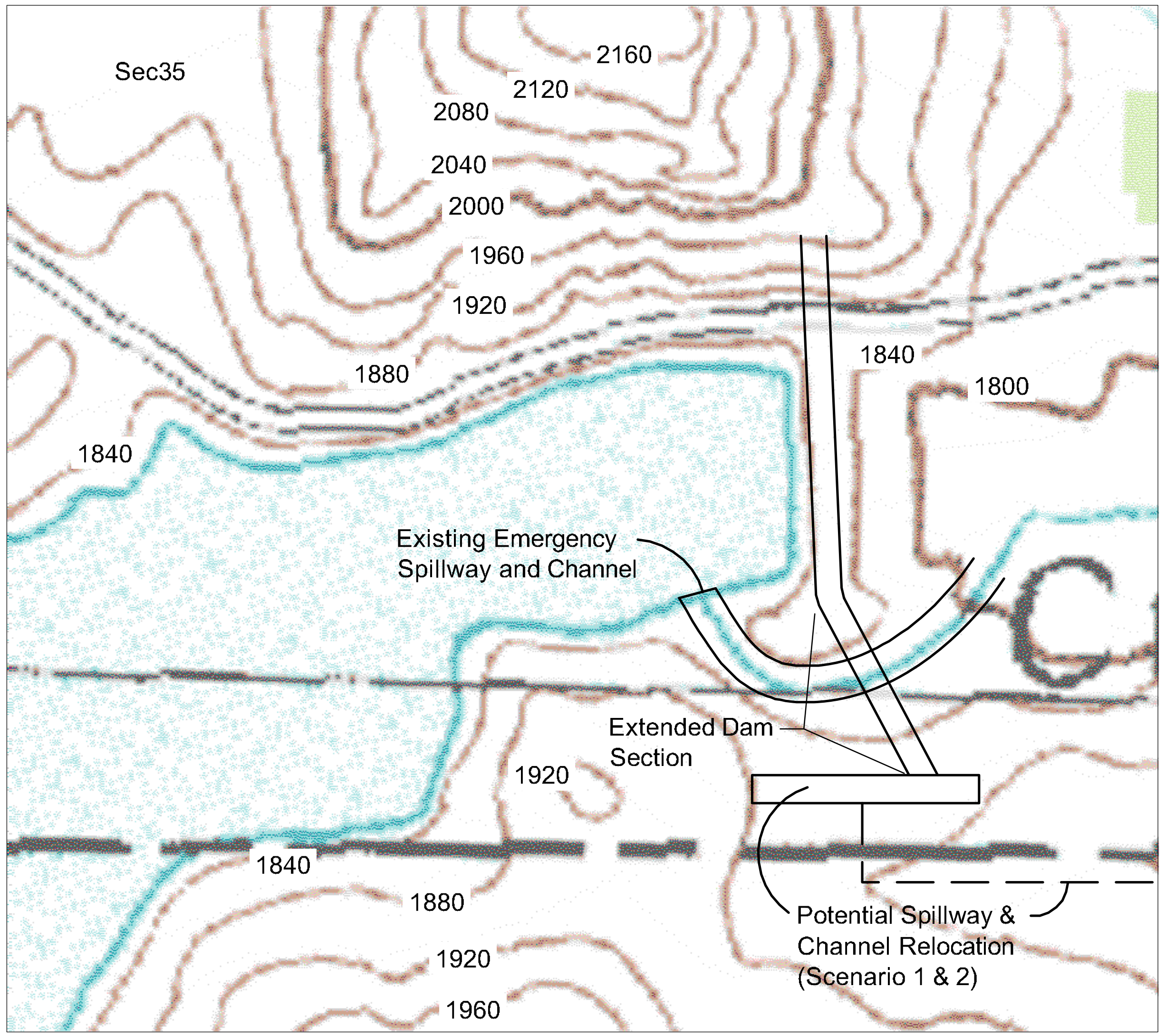
Projected Area Capacity Curve
Currant Creek Reservoir
Washington Family Ranch, Oregon

DESIGNED BY:	DRAWN BY:	DATE:	PROJECT NO.	
J. Newton	S. Schenck	FEB 2015	1163-101	FIGURE 2

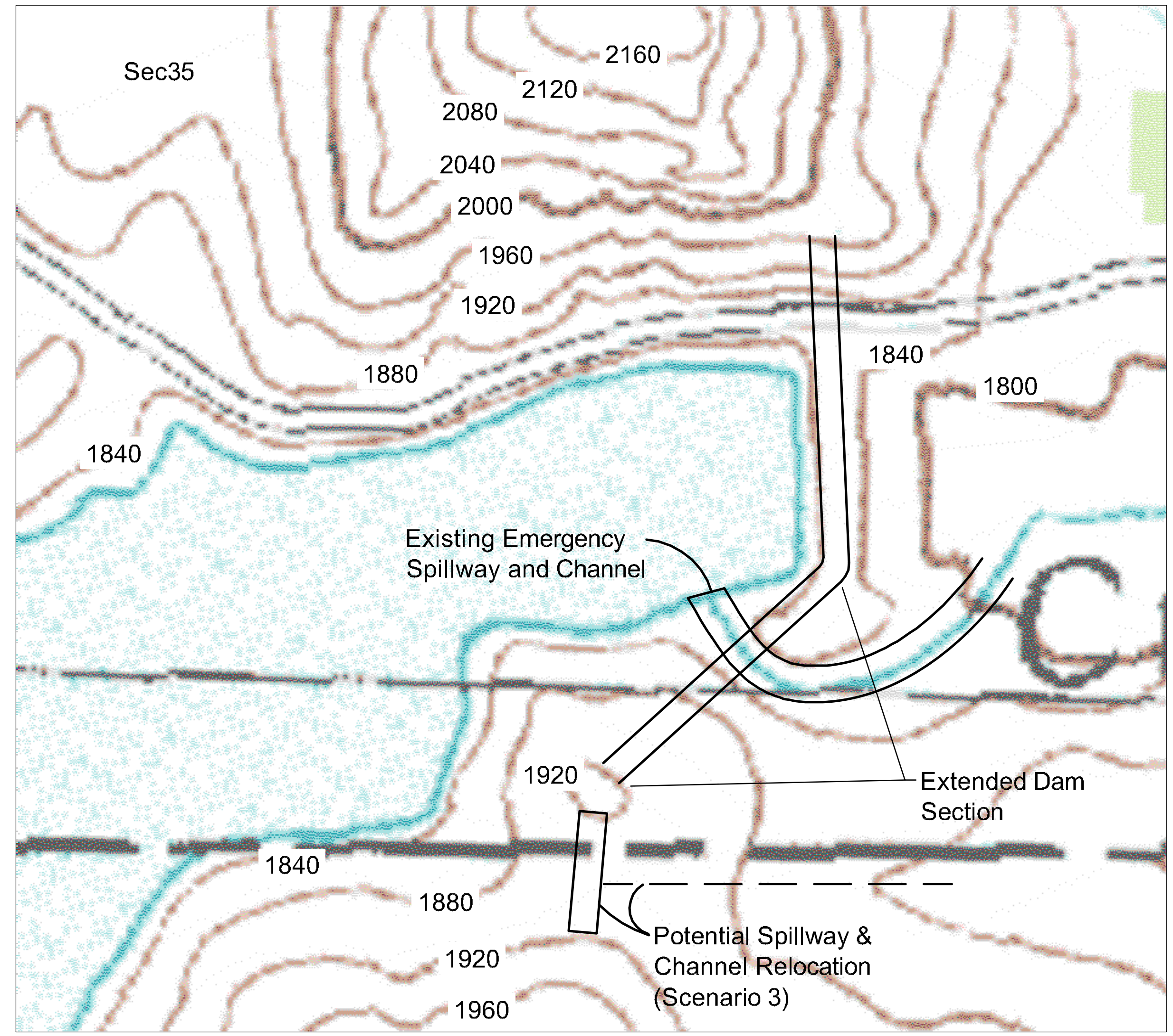
G:\1100\1163 Currant Crk Res\101\Cad\Water\W1163101_F1_ACapCrv 11:49 02/03/2015 SS

Arrastra Butte
T8S & T9S, R18E
Sec 35

Alternative I



Alternative II



0' 300' 600'
SCALE: 1" = 300'

Dam Raise Scenarios & Spillway Relocations
Currant Creek Reservoir
Washington Family Ranch, Oregon

FIGURE 3
PROJECT NO. 1163-101
DATE: FEB 2015
DRAWN BY: S. Schenck
DESIGNED BY: J. Newton

NEWTON CONSULTANTS INC.
Earth, Water and Rock Specialists
Ph: 541 504-9960 Fax: 541 504-9961

DRAFT

SITE SUITABILITY AND GEOTECHNICAL EVALUATION REPORT

FOR

WATER SUPPLY AUGMENTATION PROJECT

CURRANT CREEK DAM & SPILLWAY

PERMIT APPLICATION R-88276

JEFFERSON & WASCO COUNTY, OREGON

May 16, 2020

INTRODUCTION

The Washington Family Ranch – Young Life Camp (YLC) plans to accomplish three water supply goals: 1) increase water supply reliability for Young Life operations, including fire protection based on prior drought-period risks of inadequate supply and related detrimental impacts on Young Life operations; 2) improve stream flows and habitat conditions for fish and wildlife through flow augmentation in Currant Creek and Muddy Creek and construction of fish passage at the Muddy Creek falls, and 3) upgrade the emergency spillway according to updated hydrologic analysis, breach analysis, reduction in erosion potential, and updated OWRD dam safety standards for improved public safety and welfare.

Achievement of the three goals is planned through raising the existing spillway crest and dam to increase storage, and to reconstruct the spillway channel. The plan for these improvements is being developed in concert with the Oregon Water Resources Department, Dam Safety Section (OWRD-DSS).

Water supply for YLC and Muddy Ranch is supplied by the Currant Creek Reservoir behind the existing Currant Creek Dam. Water storage up to 1,020 acre-feet is currently authorized in the reservoir under Certificate 89219. Application R-88276 for a permit to store up to an additional 1,360 acre-feet of water is in processing at the Oregon Water Resources Department (OWRD). The actual volume of water that will actually be appropriated for storage under a new permit stemming from Application R-88276 is approximately 434 acre-feet based on refined storage planning after the new permit application was filed with the OWRD.

The dam is located in the SE ¼ SE ¼ Section 35, Township 8 South, Range 18 East, W.M., Tax Lot 1200, Wasco County, Oregon. The general location is shown on Figure 1 (Appendix A). The dam and reservoir are located on WFR-YLC property and supply nearly all of the water needs of the WFR-YLC.

Implementation of the plan to achieve the goals requires completion of a broad spectrum of engineering tasks. One of these tasks is development of a "Site Suitability/Geotechnical Evaluation" to serve as a basis for design of the elevated spillway crest and elevated dam crest. This report presents results of site suitability and engineering geology/geotechnical evaluations of the spillway area and the dam. The report includes the following major sections:

- Conclusions
- Basic Recommendations
- Description of Dam and Reservoir
- Regional Geology
- Local Geology-Dam and Spillway Area
- Geomorphology-Dam and Spillway Area
- Engineering Geology Evaluations for Proposed Raising of Spillway Control Section
- Engineering Geology Implications to Design for Spillway Crest-Raise and Spillway Channel
- Existing Dam Structure
- Basis for Design of Existing Dam and Spillway
- Seismic Analysis
- Geotechnical Implications-Storage Increase by Raising Existing Dam
- Borrow Area Locations

Appendix A: Figures 1 through **25**.
Appendix B: Photographs.
Appendix C: Table 52-4; Rock Hardness and Uniaxial Compressive Strength ("Field Procedures Guide for the Headcut Erodibility Index").
Appendix D: Century West Engineering Laboratory Test Results (1982)
Appendix E: Spreadsheet, Estimating Yield Coefficient & Seismic Slope Displacements Maximum Considered Earthquake Ground Motion Parameters.
Appendix F: STABL 6 Slope Stability Analysis Outputs
Appendix G: "Deterministic Seismic Hazard Analysis in Northwest Oregon, U.S.A.", A.G. Hull & A. Augello, 2003 Pacific Conference on Earthquake Engineering

CONCLUSIONS

Spillway

1. Additional storage under a new storage permit stemming from Application R-88276 can be provided in Currant Creek reservoir in conjunction with upgrading the emergency spillway.

2. Use of a labyrinth weir is hydraulically efficient for elevating the spillway inlet control section and gaining additional reservoir storage capacity. Efficiency is through relatively low depth of flow over the weir crest for the design runoff event. The weir crest can be elevated to increase storage capacity while providing 4.5 feet of freeboard at the dam. The flow depth over a protected embankment is much greater, requiring a lower crest elevation, which reduces storage capacity.

3. The site of the proposed weir is on the floor of the existing emergency spillway, underlain by andesitic rock and andesitic breccia bedrock, and is suitable for the proposed structure. Although the rock is subject to differential weathering and hardness varying from soft to very hard, the rock, after preparatory work, is suitable for supporting a spread or mat-type, reinforced concrete foundation for the proposed weir.

4. Eroded material in the existing emergency spillway channel consists primarily of unconsolidated colluvial materials, which overlaid andesitic bedrock and andesitic breccia bedrock. Relatively soft zones in bedrock originating from differential weathering, possibly due to local zones of more intense jointing (natural fracturing) of the rock, contributed to some bedrock erosion.

5. Exposures of bedrock after removal of overlying unconsolidated materials, has reduced the rate of erosional down-cutting in the spillway channel. The bedrock materials in the existing spillway channel are not likely to erode during single, major runoff events, including the PMF runoff event, to the point that a reservoir breach by spillway downcutting would occur.

6. Given that the bedrock conditions in the existing spillway channel are not likely to erode to a reservoir breach in a single large runoff event, inclusion of bedrock in the reconstructed spillway channel will reduce erosion potential.

7. PMF flows at 11,900 cfs through the emergency spillway will cause erosion. The spillway design objective is to pass this flow around the dam and prevent overtopping of the dam. With bedrock included as a component of the reconstructed spillway, this objective can be achieved by design of erosion protection measures based on 100 to 500-year flow conditions with the expectation some erosion and repair work can occur with these events. Although more significant erosion is likely with the PMF event, flows through the reconstructed spillway from this event are not likely to breach the reservoir.

8. Excavation of the modified spillway channel will generate earth materials in volumes greater than can be used to raise the dam, requiring disposal at some suitable location. The excess materials that are suitable for constructing an engineered fill can be placed as a toe berm against the downstream dam embankment slope, providing disposal at a very short haul distance and providing an added and beneficial stability measure for the downstream dam embankment slope. Screening is required to provide a suitable fill material for toe berm construction.

Dam Raise

1. The existing dam is planned for raising 5 feet to provide at least 4.5 feet of freeboard between the top of the dam and the maximum reservoir water elevation of 1845.0 feet under PMF runoff conditions. The existing dam is suitable for supporting the additional 5 feet of engineered embankment fill to be placed thereon.

2. Earth materials consisting of clayey silty sands to clayey, silty sandy gravels and cobbles, to clayey gravelly sands for constructing the engineered fill to raise the dam can be obtained from excavation of the modified spillway channel. With screening to remove oversize rock (larger than 6 inches), the materials are suitable for constructing the engineered fill.

3. Embankment settlement under the weight of the additional 5 feet of engineered fill will be insignificant; however, provision of 0.5 feet of camber on the top of the raised dam provides a measure of prudence.

4. Dam embankment slope stability analysis with STABL6 software according to Spencer's method (found by the U.S. Bureau of Reclamation to be suitable for dam embankment slope stability analysis) indicates that the proposed dam embankment raise of 5 feet does not bring the safety factor against rotational embankment failure below 1.5. It is feasible to raise the existing dam embankment by 5 feet with engineered fill.

5. A Magnitude 7.0 Maximum Credible Earthquake on the Warm Springs Fault, about 40 miles (64 kilometers) from Currant Creek dam is not likely to cause significant deformation to the dam. This conclusion stems from results of deformation analysis based on methods of Newmark, Bray and Travasarou, pseudostatic slope stability analysis, and cross-checking with method of Swaisgood. Potential deformation according to Bray and Travasarou yield coefficient Ky of 0.044 is 9.7 centimeters at the 16 percent probability of exceedance level, 5.0 centimeters at the 50 percent probability of exceedance level and 2.5 centimeters at the 84 percent probability of exceedance level. Potential deformation based on yield coefficient Ky of 0.20 determined from pseudostatic slope stability analysis is less than 1 centimeter.

6. Probabilistic data support Conclusion No. 5 above. Estimated ranges of peak ground acceleration based on probabilistic hazard maps ("Probabilistic Hazard Maps" included in "Earthquake Hazard Maps for Oregon," prepared by Geomatrix Consultants, Inc.) show that the range of peak ground acceleration for the dam site area is relatively low, between 0.05 and 0.10g. Probabilistic data show that shaking will be felt by most to all people and damage ranges from none to slight.

7. Ground shaking maps show the dam is located in an area with peak ground acceleration of 0.07g to 0.10g (Partner Engineering & Science, Inc., website). The USGS shows a range of 0.14g to 0.20g for an event with a 2 percent chance of occurrence in any 50-year period. The 2,500-Year Shaking Map ("Probabilistic Hazard Maps" included in "Earthquake

Hazard Maps for Oregon", Geomatrix Consultants, Inc.) shows peak ground acceleration of 0.10g for an earthquake with a 2 percent chance of occurrence in any 50-year period.

8. Following Swaisgood, the relative degree of damage would be none. Taking peak ground acceleration at 0.20g (maximum from probabilistic data shown in Conclusion No. 7 above) shows potential crest settlement of 0.058 feet, or 1.76 centimeters. Swaisgood reports that serious levels of damage were reported only in instances where the peak ground acceleration exceeded 0.2g. Investigation by Seed, Makdisi and DeAlba, 1978, found evidence "that well-built dams can withstand moderate shaking with peak accelerations up to at least 0.2g with no harmful effects."

9. Seismic analysis based on U.S. Bureau of Reclamation Design Standards No. 13, Embankment Dams, Chapter 13: Seismic Analysis and Design Phase 4: Final, and based on dam foundation material conditions, indicates that liquefaction under conditions of the Maximum Considered Earthquake and Maximum Credible Earthquake is not likely.

10. Raising the normal maximum operating elevation of the reservoir by 11.5 feet to 1,841.5 feet will increase seepage; however, the seepage rate is likely to remain very low, consistent with existing observed conditions. Seepage analysis of the existing dam by simple flow net procedures based on permeability of 0.0005 feet per day assumed in 1982 geotechnical work indicates very low average total seepage at 29 gallons per day (0.020 gpm). This estimate is consistent with existing observed conditions of low discharge primarily from the south and lessor from the middle of three toe drain pipes during spring, with no observed seepage in other areas of the dam toe, or on ground downstream of the dam. Raising the maximum reservoir operating elevation by the proposed 11.5 feet could increase the total average seepage rate to about 35 gallons per day (0.024 gpm), an increase of about 21 percent; however, the seepage rate remains very low.

11. Seepage rates for higher assumed permeability values will likely remain very low, consistent with existing observed conditions. Cross-checking seepage from the existing dam with an estimated permeability of 0.0077 feet per day based on published data for clayey sandy gravels representative of the dam embankment fill indicates a low average total seepage of 447 gallons per day (0.31 gpm). This estimate is consistent with existing observed conditions of low discharge from the dam. Raising the maximum reservoir operating elevation by the proposed 11.5 feet could increase the total average seepage rate to 543 gallons per day (0.38 gpm), an increase of 21 percent; however, the seepage rate remains low.

12. The existing toe drain and drain outlet pipe system in the dam is adequate to discharge the existing low seepage rates. The system is also adequate to discharge seepage estimated to result from raising the maximum normal operating elevation of the reservoir to 1,841.5 feet. Modifications to the downstream embankment of the existing dam must account for maintaining a functional toe drain system.

BASIC RECOMMENDATIONS

Spillway

1. The labyrinth weir proposed for the new spillway control structure should be founded on a structural, reinforced concrete mat foundation to more evenly distribute bearing pressures over a prepared bedrock bearing surface.

2. The ground surface within the existing spillway channel, within the footprint of the mat foundation, should be excavated as necessary to remove loose soil, loose rock debris and areas of relatively soft, weathered rock to provide a bearing surface on relatively hard andesitic bedrock and andesitic breccia bedrock. The excavated surface should be cleaned of loose debris to provide a clean bearing surface.

3. A vertical, reinforced concrete key wall should be extended into the ground beneath the mat foundation, to a minimum depth below the foundation bearing surface of 5 feet. The minimum width of the key wall should be 3 feet. The key wall should extend the full length of mat foundation and should extend at least 10 feet horizontally into the ground on each side of the spillway channel.

4. Seepage flow paths could be developed at the concrete-embankment interfaces between the concrete retaining walls at each end of the weir and the earth abutment materials. The retaining wall on the south end of the weir will rest against natural ground material. The retaining wall on the north end of the weir will rest against compacted, engineered fill. Cutoff and/or sand filter systems should be installed at these locations to reduce potential for seepage erosion along these interfaces.

5. The mat foundation for the weir should be designed for a maximum allowable bearing pressure of 4,000 psf applied to the prepared bedrock bearing surface.

6. The mat foundation should be designed to resist lateral forces including seismic loads, according to resistance by friction between concrete and bedrock and by passive earth resistance. Lateral resistance from friction between concrete and bedrock should be calculated with a friction coefficient of 0.45. Lateral resistance from passive earth resistance should be calculated with an allowable passive pressure of 300 pounds per cubic foot. This means that the foundation resistance to lateral movement is based on a fluid with a density of 300 pounds per cubic foot with total resistance calculated as one would calculate the pressure on a wall exerted by a fluid.

7. A stilling basin should be included in the reconstructed spillway channel downstream of the labyrinth weir to help facilitate elevation drop, reduce flow velocity and dissipate energy. For a concrete stilling basin, the allowable foundation bearing pressure, friction coefficient and passive earth resistance values in item 5 above should be used in design of the structure.

8. The excavated bearing surfaces for the weir and stilling basin shall be inspected by the Engineer of Record prior to placement of any reinforcement steel thereon.

9. The excavated key wall trench in the weir foundation and the key wall trench excavations into each side of the spillway channel shall be inspected by the Engineer of Record, prior to placement of any reinforcement steel.

10. The spillway channel subgrade (beneath rip rap) including the lower 2 feet of the spillway side channel walls should be excavated into bedrock. The excavated subgrade surface should be constructed to a reasonably uniform grade across its width and length. Excavation in rock can result in a surface with pits and protrusions. The height of protrusions above the subgrade surface should be less than 1 foot.

11. Rip rap should be used in the reconstructed spillway channel primarily to reduce erosion potential in areas of moderately soft to moderately hard bedrock that has been subjected to more intense fracturing and weathering. Rip rap should also be used to protect the lower part of the spillway sidewalls that will be subject to flows.

12. Rip rap for spillway protection shall consist of well-graded, angular to subangular, hard, durable rock materials at sizes and weights to reduce probability of erosion at flow depths and velocities consistent with the 500-year storm runoff event. The minimum specific gravity of rock for rip rap shall be 2.70. The rock shall be blended and placed to provide grading of the mass from coarse to fine, resulting in an interlocked structure of rock.

13. Excavated slopes to reconstruct the spillway shall be inclined no steeper than 1.5 horizontal to 1 vertical.

Dam Raise

1. At least 3 representative samples of earth material proposed for use in constructing the engineered fill to raise the dam shall be obtained by the Engineer of Record, or his representative, and tested by a certified materials testing laboratory for maximum dry density and optimum moisture content in accordance with AASHTO Test Method T-180 (modified proctor). These density/moisture content results shall be used in determinations of degree of fill compaction.

2. The top of dam surface to receive engineered fill shall be stripped of vegetation, crushed rock and the existing wood walkway and railing.

3. The stripped surface of the dam shall be scarified to a minimum depth of 12 inches, brought to the optimum plus 3 percent moisture content and recompacted to at least 95 percent of

the maximum dry density according to AASHTO T-180 (modified proctor). Compaction tests shall be conducted at horizontal distance intervals of no more than 100 feet.

4. The upper approximate **22** feet of the downstream dam embankment slope shall be stripped of vegetation and loose, organic soil materials. At the bottom of the stripped area, a horizontal bench shall be cut into the dam embankment slope to a minimum width of 8 feet. Fill material shall be spread over the bench area in uncompacted lifts of no more than 8 inches. The moisture content of the fill material shall be at least 3 percent and no more than 5 percent above the optimum moisture content. As the compacted fill is raised, benches should be cut at least 2 feet into the existing dam embankment to tie the new fill into the existing fill. The fill placement, compaction and benching process shall continue to the top of the dam. The slope of this new fill shall be inclined no steeper than 1.5 to 1. Compaction tests shall be performed at the top of each 2-foot thick layer of compacted fill. Compaction tests shall be performed at horizontal spacings not to exceed 100 feet.

5. The upper section of the upstream dam embankment should be extended from the existing top of the slope upward, to the proposed new top of the dam. The slope angle for this new fill should match the 3 to 1 slope angle of the existing upstream dam slope.

6. Excess excavated earth materials from spillway construction shall be placed along the toe of the existing downstream dam embankment slope. The intent is to dispose of excess material through construction of a non-structural toe berm. Prior to any placement of fill in this area, the existing ground shall be stripped of vegetation, organic material and loose organic topsoils. Care shall be taken to avoid damage to the existing PVC toe drain outlet pipes emerging from the toe of the dam embankment.

7. The stripped ground in the toe area of the downstream dam embankment slope shall be inspected by the Engineer of Record or his representative prior to placement of any fill thereon.

8. The stripped ground surface in the area to receive fill shall be scarified to a minimum depth of 12 inches, brought to the optimum plus 3 percent moisture content and recompacted to at least 95 percent of the maximum dry density according to AASHTO T-180 (modified proctor).

9. A trench drain system should be constructed in the natural ground of the proposed toe berm area. The trench drain should interconnect with the existing gravel/sand toe drain within the toe area of the dam and with the existing three toe drain discharge pipes. The toe drain should be constructed, tentatively, with sand meeting the ODOT gradation specifications in Table 02690-5 of the 2018 Standard Specifications for Portland concrete fine aggregate. Preliminary analysis with soil gradation data from the 1982 geotechnical investigations indicates the referenced sand potentially meets filtration and permeability criteria. The existing PVC discharge pipes from the dam toe drain shall be extended with

additional drain pipes to new discharge points located outside the toe of the new embankment to allow free, unobstructed discharge and full visibility for future monitoring.

10. Sand meeting the above-referenced ODOT gradation for the trench drain system is a tentative consideration for both filtration and permeability needs based on preliminary analysis of soil gradation results from 1982 testing. The sand gradation for these needs shall be verified with gradation tests on fill materials as they are being screened for use as toe berm embankment fill. The gradation shall also be verified with gradation tests on natural ground soils into which the trench drain system will be installed.

11. The drain pipe to be installed in the trench drain should consist of ADS perforated, heavy duty (highway grade) pipe. The minimum diameter should be 6 inches. Perforations, including slots, shall be no wider than **3** millimeters. The perforation width shall be verified according to gradation tests on filter sand and slot-sizing based on filter criteria.

12. The existing three toe drain discharge pipes exposed at the toe of the downstream dam embankment shall be extended and connected into the new perforated trench drain pipe. Pipe for these extensions can match the diameter of the existing pipes, and should consist of ADS heavy duty (highway grade), non-perforated pipe. Connections shall be with water-tight connectors.

13. Three new drain pipes consisting of ADS non-perforated, heavy duty (highway grade), pipe 6 inches in diameter shall connect to the trench drain pipe, and extend beneath the toe berm to unobstructed discharge points outside the toe of the toe berm fill.

14. Fill materials shall be placed with uncompacted lifts no thicker than 8 inches spread across the area to be filled. The moisture content of the fill shall be at least 3 percent and no more than 5 percent above the optimum moisture content. Each lift of fill shall be compacted to at least 85 percent of the maximum dry density in accordance with AASHTO Test Method T-180 (modified proctor). The finished toe berm is not intended to be a structural engineered fill and the reduced compaction is to provide guidance in constructing the fill to a uniformly firm condition. Compaction tests shall be performed at vertical elevation intervals of no more than 3 feet and at horizontal spacings of no more than 100 feet.

15. Compaction testing shall be done by a certified material testing laboratory. Compaction test results shall be provided to the contractor on the day they were performed and to the Engineer of Record the same day they were performed.

16. At least two piezometers shall be constructed at locations on the upper part of the upstream dam embankment slope. The piezometers shall be constructed to a minimum depth below the ground surface of 80 feet.

DESCRIPTION OF DAM AND RESERVOIR

The dam was designed and constructed in 1982-83 under the oversight of David Newton, P.E., and C.E.G. through Century West Engineering Corporation, Inc. of Bend, Oregon. The construction drawings and specifications were approved by Mr. Al Petska, OWRD Dam Safety Engineer. The dam was constructed as an engineered, homogeneous earth fill structure with a maximum height of 80 feet. The upstream slope of the dam embankment is inclined at 3 horizontal to 1 vertical and the downstream slope is inclined at 2 horizontal to 1 vertical. A filtered toe drain system was constructed within the dam.

Dam construction was by Neo Rajneesh International Commune personnel with equipment provided by a contactor located in Washington State. The soil materials used to construct the embankment were placed in uncompacted lifts 6 to 12 inches in thickness and compacted to at least 90 percent of the maximum dry density (modified proctor, AASHTO T-180), at or near the optimum moisture content. Construction of the dam and compaction testing was done under the oversight of David Newton and Century West Engineering Corporation (CWEC) staff. CWEC staff also provided construction staking. Staff were housed at the Commune during the construction

The storage permit issued by the OWRD authorizes up to 1,020 acre-feet of storage in the reservoir. The reservoir surface area is 44 acres at the maximum normal operating pool elevation of 1,831 feet. The watershed area discharging to the reservoir is approximately 35 square miles. The watershed area is shown on Figure 2.

The dam is constructed across Currant Creek, a tributary of Muddy Creek, which is tributary to the John Day River. The distance from the dam to the confluence of Currant and Muddy Creeks is about 2.75 miles. The distance from the dam to the John Day River is about 5.8 miles.

The dam is located about 1.2 stream miles upstream of the nearest buildings and about 2 stream miles upstream of the original Rajneesh headquarters building, all near the airstrip. Other residential and operations facilities are located about 2.75 stream miles downstream of the dam. Figure 3 shows the relationship of the dam and reservoir to downstream infrastructure and the John Day River.

A High Hazard classification was assigned to the dam by the OWRD-DSS at the time of its original design. Accordingly, the emergency spillway was designed to pass the one-half Thunderstorm Probable Maximum Flood as stipulated by the OWRD-DSD at that time. The flow rate for this event was estimated at 27,800 cubic feet per second. The existing spillway was designed to pass this flow rate with 2 feet of freeboard at the dam.

Function of the dam has been without incident over its life span to date. No slumping of embankment slopes or observable settlement of the crest has occurred. Occasional, short-term water discharge occurs from the southern toe drain discharge pipe in the spring season. No evidence of seepage through the dam and discharge from the downstream embankment slope has been observed.

REGIONAL GEOLOGY

The Geologic Map of Oregon (George W. Walker and Norman S. MacLeod, 1991, USGS) shows that the geologic platform for the dam and reservoir site is comprised primarily of consolidated sedimentary materials and rock. The dominant geologic formation (Tca) extending away from the site to rough distances of 3 miles north, 4 miles west, 35 miles northeast and 40 miles south is described as clastic rocks (rock or sediment composed principally of fragments derived from pre-existing rocks and transported from their place of origin) and andesite flows, mostly of the Clarno Formation of central Oregon and un-named rocks of the Basin and Range Province of south-central Oregon.

The Tca rocks are mostly andesitic lava flows, domes, breccia and small intrusive masses and lesser basaltic to rhyolitic rocks. Also included are interlayered saprolite, bedded volcaniclastic and epiclastic mudstone, claystone, siltstone, sandstone, conglomerate and mudflow (lahar) deposits. The rocks are aged at 54 to 37 million years old and are associated with lower Oligocene (possibly), Eocene and Paleocene (possibly).

Relatively small, local areas of tuffaceous rocks (Tct) are mapped within the greater extent of the above Clarno Formation rocks (Tca). These rocks are described as predominantly tuffaceous facies of the Clarno Formation, of lower Oligocene (possibly) and Eocene age.

Within the larger map area of the above-described Clarno Formation rocks is an area of approximately 60 square miles mapped as Quaternary landslide terrane. The nearest boundary of this terrane is approximately 2 miles north of the dam and reservoir site. It is likely that landslide activity occurred in the above-described Clarno Formation rocks, particularly the claystone or tuffaceous materials. A smaller, localized landslide is mapped to the north edge of Muddy Road north of the reservoir as shown in the top center of Figure 9. The shortest distance between the mapped landslide and the reservoir is about 100 feet (based on maximum reservoir water surface elevation of 1,831 feet). A round-shaped "massif" of andesite breccia is mapped between the landslide and the reservoir. The landslide was described in the 1982 geotechnical investigation report as a probable Pleistocene event occurring in a claystone unit (Robinson, 1975). The report concludes that it has since stabilized through extensive erosion in the upper portions of the slide mass.

The northern boundary of the Clarno Formation map area is marked by its contact with the mapped area of the Grand Ronde Basalt (Tcg). The contact with the basalt is about 10 miles north of the dam and reservoir site. The mapped area of the basalt extends westward of the dam and reservoir site about 20 miles and over 100 miles eastward, in a band about 20 miles wide.

Rocks of the John Day Formation are mapped about 5 miles west of the dam and reservoir site. The mapped area extends westward and southwesterly.

LOCAL GEOLOGY – DAM AND SPILLWAY SITE AREA

Overview – Site Area

Geologic conditions for areas more proximal to the dam and reservoir site described below are from the geologic investigations conducted in 1982 for design of the existing dam. Updates are included later in this report based on additional engineering geology and geomorphic site evaluations conducted in May, June and November, 2019. The purpose of these evaluations is to provide design criteria for increasing water storage in the reservoir through construction of a new spillway control section and raising of the existing dam. These evaluations are described below, following the regional and local geology sections of this report.

The Currant Creek Dam and spillway are situated in a geologically complex area of ancient volcanic activity. Geologic materials in the area are associated primarily with the John Day and Clarno Formations of the Tertiary Period ("Reconnaissance Geologic Map of the John Day Formation in the Southwestern Part of the Blue Mountains and Adjacent Areas, North-Central Oregon," Paul T. Robinson, 1975, USGS Miscellaneous Investigations Series Map I-872). Some local basalt outcrops of Columbia River Basalt Group also occur. A general regional geologic map is shown on Figure 4. A regional geologic map on Google Earth imagery of the area around the dam and spillway is shown on Figure 4A. This figure reveals andesite flows and andesite breccias in the dam and spillway area. No definitive linear structures are apparent on the imagery in the area of the dam and spillway.

The John Day Formation is 39 million to 18 million years old. The Clarno formation is 58 to 39 million years old and is overlain by the John Day Formation. These formation materials have been subjected to faulting, folding and extensive weathering. Three major bedrock units that have been identified in the general dam and spillway area as described below.

The dominant rock types mapped (Robinson, 1975) in the Currant Creek dam and reservoir area are olivine andesites (Tcl) of the Clarno Formation and bedded volcaniclastics, tuff, ash flow tuff, sandstone, siltstone and conglomerate (Tct) of the Clarno Formation. Phyllite and sedimentary rocks, slates and graywackes (pT) overlain by the Clarno Formation materials are mapped in an area east of the reservoir. Welded ash flows (Tja) of the younger John Day Formation are mapped in local areas northwest of the reservoir. An anticline/syncline fold system is mapped in the reservoir area. The axis of the anticline and the syncline trends northeast and both plunge to the northeast. Their mapped trace extends from the southwest to the area of the reservoir.

The andesites mapped in the dam and reservoir area are described (Robinson, 1975) to occur as andesite flows and domes, occurring with volcanic flow breccias. Flow breccias were observed in the area of the dam and reservoir as caps on higher ridge tops. Volcanic breccia overlays the andesite and bedded volcaniclastic materials in some areas near the dam. The contact between the breccia flow and the underlying units is an unconformity, meaning that the breccia was deposited on an erosional surface of a different geologic age (the sequence of deposition over time was interrupted by a non-depositional period of erosion before the breccia was deposited on the eroded surface). The breccia flow appears to have filled in areas of relatively low relief. Breccia flow materials are relatively hard. Underlying, relatively soft materials are more easily eroded, such that the more resistant breccia's better withstand erosion and remain to form higher ridges and peaks.

Very hard, relatively unweathered andesite is exposed in the road cut for the Big Muddy Road, passing along the north side of the reservoir and the dam. Very hard, relatively unweathered andesite comprises the bottom of the 40-feet wide core trench beneath the central part of the dam. Andesite observed in areas of the spillway is subject to differential weathering reflected in moderately hard rock to very hard rock conditions. More detail is described in the engineering geology evaluations later in this report.

The andesite is fractured by a number of joint sets, oriented at various strike and dip directions. A principal joint set exposed in the roadcut strikes approximately east-northeast and dips southeast at roughly 40 to 45 degrees. Fractures in the andesite exposed in the foundation core trench were generally tight and were hand-grouted by Rajneesh personnel prior to placement of any dam embankment fill thereon.

Geologic mapping ("Geologic Map of Oregon," Walker & MacLeod, 1991, USGS) shows locations of mapped and inferred faults. A mapped fault is shown over a length of about 6 miles, passing through the approximate reservoir area. The fault trend is northeast-southwest. No age is assigned to the fault and the fault is not mapped as an active fault (Figures 10, 11 and 12).

The dominant soils in the Currant Creek watershed are clays, silty clays and clayey silts. The clay soils originate from weathering of the volcanic tuffs, ash flow tuffs and welded tuffs of the John Day and Clarno Formations that dominate the watershed area. The clay materials are transported by runoff and comprise minor to major fractions of the matrix material in coarse-grained alluvial sands and gravels.

North Dam Abutment

Geology of the dam and spillway area is shown on Figure 5. Olivine andesitic rock and volcanic breccia units underlay both abutments of the existing dam. Olivine andesite is the predominant rock unit on the north abutment. This rock forms the high peak north of the dam and is exposed in the road cut for the Big Muddy Road which passes by the north side of the dam and reservoir. Exposures of this rock are gray to brown in color and indicate the rock is visually fresh with minimal alteration along fractures.

The rock is fractured with fracture spacing ranging from 1 to 3 inches. This range of fracture spacing best fits the "very close spacing" classification according to criteria recommended by the International Society for Rock Mechanics (ISRM). Fracture spacing in this classification ranges from 0.02 to 0.06 meters, or 0.79 to 2.4 inches. The dominant fracture orientation appears to align with a strike of North 80 degrees East, with a 40 to 50-degree dip to the south.

Observed fractures are relatively tight. Outcrop observations indicate that the intensity of more open fractures decreases toward the interior of the rock mass. Considering that evidence of sliding movement along fractures is not clear, it is reasonable to conclude that the majority of fractures are joints. Joints are generally caused by tension stresses and displacement is by movement of rock on one side of the joint away from the other side, with no sliding displacement between rock on each side of the joint. Some observed fractures reveal evidence of some sliding movement between rock on each side of the fracture. These cases are consistent with faulting.

Core Trench for Existing Dam

A core trench was excavated in the alluvium for the existing dam. The trench was constructed to depth of 15 to 20 feet, side slopes of 2 horizontal to one vertical, bottom width of 40 feet and top width of 70 feet. The bottom of the trench is comprised of hard, relatively fresh, fractured andesite. Fractures in the andesite were relatively tight; however, they were hand-filled with brushed-in grout prior to any placement of embankment fill thereon. The presence of andesite in the core trench reveals its extent from the north abutment and beneath the dam.

South Dam Abutment

Volcanic breccia appears to be the more predominant rock type in the south dam abutment. Breccia exposures are gray, visually fresh and massive, with a few fractures in random orientation through the outcrops. Breccia exposed in exploratory excavations was yellow-brown, pale orange brown and gray in color. The breccia appears to be dacitic with angular gravel to boulder-size clasts of varying volcanic composition.

The andesite rock unit was mapped on the south abutment below the approximate mid-height of the dam, extending westward away from the dam and downward, disappearing beneath alluvium on the floor of the Currant Creek floodplain. The contact between the andesite and the breccia unit was observed on the south abutment as shown on Figure 5. Generally, this contact between the two rock units very sharp with some slight, low-grade metamorphism extending about 6 to 12 inches into the andesite. No voids or fractures were observed in association with this contact.

Currant Creek Floodplain

The Currant Creek floodplain at the dam site was roughly 100 to 150 feet wide prior to construction of the dam. The floodplain was underlain by quaternary period alluvium consisting generally of loose to medium dense, silts, clayey sands, silty sands, sandy gravels, clayey gravels, sand/gravel/cobble mixes and clay. Some amount of clay appears to occur in most of the granular materials.

The thickness of alluvium at a cross-section at the dam site ranges from about 14 to 25 feet based on exploratory boring findings. The alluvial materials overlay hard, dense andesite. The core trench for the existing dam was excavated to the andesite bedrock, fully penetrating the alluvium.

Landslides

A relatively old landslide is mapped (Robinson, 1975) in the area mapped (Robinson 1975) as andesite (Tcl) northwest of the existing dam as shown on Figure 5. The landslide appears to have been active in the Pleistocene (2.58 million to 12,000 years ago). It appears to have stabilized through extensive erosion in the upper portions of the slide mass.

Additional research of potential landslide conditions was done by review of the "Landslide Susceptibility Map of Oregon," 2016, prepared by the State of Oregon, Department of Geology and Mineral Industries. No areas of landslide susceptibility were found on the map for the area of the dam and reservoir.

GEOMORPHOLOGY – DAM AND SPILLWAY AREA

The evolution of topographic features of the dam and reservoir area appears tied primarily to deformation and erosion of the various geologic units of the site area. Present terrain features are due primarily to differential erosion. Geomorphic conditions are illustrated on Figure 6.

Terrain in the dam and reservoir area consists of drainages dissecting areas of higher ridges and buttes. Drainage patterns do not show a clear, conspicuous relationship to geologic structure such as bedding, faulting, etc. Many of the higher-elevation hills and buttes are characterized by rounded tops and moderately steep slope angles. These characteristics are due to relatively soft geologic materials of which they are composed and relative ease at which they are weathered. These materials generally consist of volcanic ash, tuff, and lapilli tuffs.

Prominent, relatively rough, higher-elevation ridges in the area remain due to lessor degrees of erodibility of the geologic materials of which they are composed. Andesite and andesite breccias, and some welded tuffaceous rocks are exposed in the more prominent ridges and peaks. These materials are more resistant to weathering and now stand in relatively high relief.

Through weathering processes, softer geologic materials have been cut down and eroded away. Harder geologic materials less prone to weathering stand in higher relief, forming ridge tops and prominent peaks.

One landslide is mapped (Robinson 1975) at an approximate location shown on Figure 6, a short distance northwest of the dam site, outside the existing and proposed reservoir area. This landslide was also acknowledged and mapped by CWEC during its site geologic mapping for the original dam design work in 1982. No other landslide terrain has been confirmed in the dam and reservoir site area during current site investigations.

ENGINEERING GEOLOGY EVALUATIONS FOR PROPOSED RAISING OF SPILLWAY CONTROL SECTION

Purpose

Engineering geology conditions of earth materials in the spillway area must be accounted for in design of a structure to raise the spillway crest elevation. Foundation materials must be suitable for support of the crest-raise structure. Erosion potential of earth materials in the spillway channel area must be accounted for in order that appropriate erosion control provisions are included in the crest-raise project. The purpose of the engineering geology evaluations is to clarify suitability of foundation materials to support the crest-raise structure and to clarify erosion potential of geologic materials subject to spillway discharge flows.

Field Evaluations

Field evaluations were conducted by David Newton, Oregon registered civil engineer and engineering geologist. The evaluations included observations of earth materials exposed in six exploratory backhoe pits excavated near the inlet of the existing spillway. The approximate locations of the pits are shown on the attached Figure 5. This work was conducted on May 23, 2019.

Field evaluations of the existing dam and spillway areas were also conducted by David Newton in June and November, 2019. These evaluations were conducted by field observations of terrain conditions and outcropping geologic units in the near proximity of the dam and spillway. The purpose of the evaluations was to investigate the type and distribution of geologic materials in the dam and spillway areas and their engineering properties relative to design of the spillway crest-raise and dam-raise structures, and design of erosion control provisions for the spillway.

Evaluation Findings

Reconnaissance Observations

The terrain in the spillway area of the dam includes a northeasterly-trending, short ridge capped with relatively hard rock. Observation of the rock exposures indicates the rock consists of andesitic breccia. The rock is igneous in classification and is fine-grained with inclusions of older, angular rock fragments. This ridge structure forms the south abutment of the dam and the north side of the existing spillway channel. Flow through the spillway is held away from the dam by this ridge.

The inlet of the spillway consists of a concrete apron constructed when the original dam was built. The inlet part of the spillway downstream from the concrete apron is relatively flat and up to about 200 feet wide. The spillway channel curves from a southeasterly flow direction in the entrance area, to a northeasterly flow direction extending to its confluence with Currant Creek downstream of the dam.

A relatively narrow erosion channel has formed along the southerly edge of the spillway flow channel. This channel, shown on Figure 5, is about 5 to 8 feet deep and has been filled with rock and concrete debris by YLC to retard erosion. Downstream of the spillway inlet area, erosion has formed a head-cut with a vertical elevation drop of approximately 6 to 8 feet over a horizontal distance of roughly 8 feet. A profile along the erosion channel is shown on Figure 5A.

Andesite and andesitic breccia exposed in the flow channel near the spillway inlet and at locations downstream, near and in the head-cut area (Figure 5 and 5A), is hard to very hard based on rock hammer tests (observing effects of rock hammer impact on rock fragments is a practical field method for classifying rock according to its uniaxial compressive strength, "Field Procedures Guide for Determining Headcut Erodibility Index, Part 628, National Engineering Handbook"). Very few joints were observed in these exposures, revealing the massive character of this rock in these exposures.

The flow channel in which hard to very hard rock was observed results from erosion by spillway flows that have occurred since the late 1980's. The channel appears to have been eroded into zones of softer rock. It is possible that softer rock material exists as an outer "rind" of more weathered and fractured rock that grades into much harder rock at depth. Part of the erosion appears to have removed the softer "rind," leaving the harder, un-eroded rock exposed in the channel. It also appears that spillway flows eroded zones of more weathered and softer rock that exist within the harder rock mass. This results from differential erosion, possibly reflecting a higher degree of weathering in rock zones containing concentrations of fractures, including jointing and possible faulting.

The headcut (Figure 5, 5A) exposes hard andesite and andesite breccia. This rock was originally overlain by unconsolidated colluvial and alluvial silts, sands, gravels, cobbles and boulders. These unconsolidated materials were eroded away by spillway flows, exposing the underlying, hard to very hard rock in the headcut and on the bottom of the erosion channel upstream of the headcut.

Exploratory Backhoe Pit Observations

Exploratory backhoe pits were excavated in the general spillway inlet area with a Cat 316F excavator equipped with a 24-inch wide bucket. Weathered rock was encountered at depths of about 1 to 2 feet below a surface layer of loose, moist, dark brown silty to clayey sands with gravel. The weathered rock exposed in the pits varied in color from gray, gray brown, yellow to orange brown, to pale green. The rock broke out of the pits in angular, blocky to rectangular fragments ranging from about 2 inches to 12 inches in size. This breakage characteristic is controlled by the insitu fracture patterns in the rock. Rock conditions in the pits are summarized below:

> Pit 1: Encountered gray to gray brown rock was hard and fragments could not be broken with a rock hammer. Broke out into 4 to 12-inch blocks, reflecting shape control by insitu fracturing. Rock appears to be andesite. Excavation stopped at depth of 2.2 feet. More than one blow with a rock hammer was required to break rock chunks. Impact of the pointed end of the hammer made no distinctive depression on the rock.

Pit 2: Encountered gray to gray brown rock was hard, breaking out into angular, blocky to elongate fragments with orange brown staining on fracture surfaces. Shape of fragments is controlled by insitu fracturing. The rock appears to be weathered andesite and weathered flow breccia. Excavation stopped at depth of 1.5 feet. More than one blow with a rock hammer was required to break rock chunks. Impact of the pointed end of the hammer made no distinctive depression on the rock.

Pit 3: Encountered yellow to orange brown weathered andesite (to andesitic breccia). The rock is more weathered than rock in Pits 1 and 2 to a depth of about 2.5 feet. Rock breaks out into rubble, 2 to 12-inch, blocky to tabular fragments that are easier to break with rock hammer than fragments of Pit 1 and 2. Fragment shapes controlled by insitu fracturing. At 2.5-foot depth, becomes harder to excavate. Rock is more massive. Rock hammer impact leaves 1/8-inch dent (square end of hammer). Excavation stopped at depth of 3.4 feet. One blow with a rock hammer broke rock chunks. Impact of the pointed end of the hammer made 1 to 3 mm indentations on rock surfaces.

Pit 4: Encountered orange brown to yellow brown, hard weathered andesite. Harder than rock in Pit 3 (rock in Pit 3 was softest of all). The rock is fractured and breaks out in blocky to somewhat elongate, angular fragments in approximate size range of 2 to 12 inches. Fragment shape is controlled by insitu fracturing. Fragments are harder to break with rock hammer than those in Pit 3. Quartz veins encased in a black, wet material were observed. Excavation was stopped at a depth of 2 feet. More than one blow with a rock hammer was required to break rock chunks. Impact of the pointed end of the hammer made no distinctive depression on the rock.

Pit 5: Rubbly, loose, fractured weathered andesite that breaks out in 1 to 3-inch, angular, blocky fragments encountered a depth of 6 inches to about 2 feet. Below a 2-foot depth, orange brown, gray brown to pale green weathered andesite with hardness similar to that in Pit 3 was encountered. Hammer impact left 1/16-inch dent. Rock breaks out in 1 to 12-inch blocky to elongated, angular fragments. Shape of fragments is controlled by insitu fracturing of the rock. Rock was hard to excavate. Excavation stopped a depth of 2.5 feet. More than one blow with a rock hammer was required to break rock chunks. Impact of the pointed end of the hammer made no distinctive depression on the rock.

In summary, the encountered rock appears to be of andesitic and breccia composition. Although the exposed rock is relatively hard, some variability in hardness was observed that appears to correlate with degree of weathering. Photographs (Appendix B) show the character of rock in some of the pits. Examination of rock exposed in the pits suggests that the material has reasonable resistance to erosion. It is noteworthy that spillway discharge concentrated along the south side of the spillway channel over many years has eroded a channel that is 5 to 8 feet deep. This indicates that the weathered and fractured condition of some rock zones in the spillway channel increases susceptibility to erosion.

ENGINEERING GEOLOGY IMPLICATIONS TO DESIGN FOR SPILLWAY CREST-RAISE AND SPILLWAY CHANNEL

Observations discussed above of rock exposed in the backhoe pits reflected inconsistency with the channel erosion and headcut located downstream of the spillway inlet area. Accordingly, field evaluations shifted to the eroded channel area and other areas within and outside the eroded channel area to search for relationships between geology and the more extensive channel erosion.

Observations of a downcut area of the existing spillway near the headcut as shown on Figure 5 and 5A revealed relatively hard, light to moderately weathered rock exposed on the bottom of the channel. This relatively erosion-resistant rock covered the channel bottom and extending up the channel sidewalls for a height of less than one foot. Materials above the exposed rock consist of unconsolidated colluvial and alluvial sands, gravels, cobbles and boulders. Light to moderately weathered bedrock was also observed in the headcut. The vertical drop of this feature is about 6 to 8 feet located as shown on Figure 5 and 5A. This rock is andesite and andesite breccia, similar to that exposed on the eroded channel bottom upstream of the headcut, and similar to rock exposed in outcrops flanking the channel and exposed in the exploratory backhoe pits near the spillway inlet. The exposure of the rock in the near-vertical headcut reflects its ability to resist erosion.

The observations of colluvial and alluvial materials overlying relatively hard bedrock suggest that the erosion channel cut in the spillway and the headcut were created by erosion of unconsolidated colluvial materials and relatively soft bedrock materials overlaying much harder, more erosion-resistant bedrock material. Once the more erodible materials were removed, the underlying weathered bedrock offered more resistance to erosion and downcutting occurred to a much lower degree.

The presence of light to moderately weathered andesite and andesite breccia exposed in backhoe pits in spillway inlet area and exposed in the erosion channel and headcut areas described above, suggest that the andesitic rock could also underlie the area of the spillway channel downstream of the inlet. If this is the case, the bedrock material could provide a relatively erosion resistant bottom and sidewalls for a new, reconstructed spillway channel. Accordingly, field observations for bedrock exposures were made over an expanded area around the existing spillway channel.

Figure 5 shows approximate locations of bedrock exposures observed during the field observations. The locations are mapped on Google Earth imagery. The bedrock material appears to be andesitic, similar to that exposed in the spillway inlet area. Based on these observations and mapped distribution of bedrock exposures, it is concluded that the bedrock material can be used as a subgrade for the bottom and partial sidewalls of a reconstructed spillway.

The headcut formed by removal of erodible materials presents a significant elevation drop that must be accounted for in the new spillway channel. Excavation to remove erodible material and cut the spillway bottom and sidewall subgrades can also create a flatter grade through the vertical drop in the headcut area.

Although bedrock is present that can retard erosion, some erosion protection should be placed on the subgrade for the spillway bottom and subgrade for spillway sidewalls. Differential weathering of the bedrock resulting in local relatively soft bedrock conditions can result in local areas of spillway erosion. However, based on field observations and the concept described above, it appears that the reconstructed spillway will perform with less erosion potential than the original spillway.

Engineering Properties of Rock for Spillway Control Section and Channel

Engineering properties of the observed andesitic and breccia rock in the spillway channel and outcrops were characterized relative to suitability for foundation of the crest-raise structure and spillway function. Characterization was based on criteria in the publication "Chapter 52, Field Procedures Guide for the Headcut Erodibility Index," Part 628 Dams National Engineering Handbook, United States Department of Agriculture, Natural Resources Conservation Service, Revised March 2001.

The above-referenced Guide describes rock conditions and criteria for estimating uniaxial compressive strength of rock and for determining the Headcut Erodibility Index (Kh). Rock compressive strength is useful in assessing suitability of the rock as foundation material for the crest-raise structure and for determining the Kh value.

The Kh value is used with a calculation of Stream Power to determine potential for rock erosion by streamflow. Equations for calculation of Kh and Stream Power are below.

> Kh = (Ms)(Kb)(Kd)(Js) where:
>
> Ms is uniaxial compressive strength of rock (MPa), Kb is particle or fragment size of the rock blocks, Kd is interblock strength and Js is relative shape and orientation of blocks.
>
> Stream Power = (W)(V)(H)(S) where:
>
> W is unit weight of water, V is velocity of flow, H is depth of flow and S is the hydraulic grade line slope, all in metric units.

The various applicable rock conditions for Kh determination are discussed below.

Rock Hardness and Uniaxial Compressive Strength

Table 52-4 from "Field Procedures Guide for the Headcut Erodibility Index", included in Appendix C of this report, presents correlations between rock material hardness and uniaxial compressive strength, and field identification tests for determining rock material hardness. A note in the Guide in regard to field identification tests and use of the table states that "Because large differences in rock strength are required to appreciably affect the headcut erodibility index, the precision afforded by expensive laboratory tests is rarely justified. Experience shows that conducting field estimates of rock material hardness is a practical way of obtaining adequate assessments of strength."

Field identification tests according to the above-cited Guide on rock chunks from the 5 exploratory pits in the spillway channel indicate rock hardness ranges from moderately soft to hard rock.

Light blows of the pointed end of a rock pit applied to rock chunks from exploratory Pit 3 left shallow indentations roughly 1 to 3 mm deep. This finding indicates that the rock in Pit 3 is moderately soft rock. The range of uniaxial compressive strength for this rock condition is shown on Table 52.4 at 5.0 to 12.5 MPa (725 to 1813 psi). During excavation, rock fragments broke out on along fracture surfaces, indicating that some jointing exists in this rock. Rock fragment sizes up to prevalent maximum of 8 inches and some up to about 12 inches suggest prevalent joint spacing in a 4 to 8-inch range, although evidence of closer spacing was observed in some cases.

Breakage of rock chunks from exploratory Pit 1, 2, 4 and 5 required more than one hammer blow for breakage. These findings indicate that the rock in these pits is hard rock. The range of uniaxial compressive strength for this rock condition is shown on Table 52.4 at 50.0 to 100.0 MPa (7,252 to 14,504 psi). Rock chunks revealed joint surfaces which control chunk sizes as they are broken out by excavation. It appears that prevalent joint spacing ranges generally from about 4 to 8 inches.

Andesite and andesitic breccia exposed in the flow channel near the spillway inlet and at locations downstream, near and in the head-cut area (Figure 5 and 5A), is very hard to hard, based on rock hammer tests. Very few joints were observed in these exposures, revealing the massive character of this rock in these exposures. Uniaxial compressive strength for this rock is shown on Table 52.4 to range from 50 to 250 MPa (7,252 to 36,259 psi).

Rock in the spillway channel area ranges in hardness from moderately hard to very hard, to moderately soft. Accordingly, uniaxial compressive strength values for estimating erosion potential were selected for moderately hard rock and very hard rock. These values are below:

Ms for moderately hard rock = 12.5 MPa (1,813 psi)
Ms for Very hard rock taken at low end of range = 50 MPa (7,252 psi)

Block/Particle Size Number (Kb)

The primary method to calculate Kb is: Kb = RQD/Jn

Where: RQD = rock quality designation and Jn = joint set number.

RQD represents the sum of the length of core pieces greater than 0.1 meter divided by the total core run length (generally 1.5 meters), expressed in percent. Core sampling was not performed and RQD is estimated from criteria based on number of joints in a cubic meter of rock (Chapter 52, Field Procedures Guide for the Headcut Erodibility Index).

Kb for moderately soft rock is based on an estimated 20 joints per cubic meter of rock, the RQD is estimated at 50 percent based on field observations of rock exposed in the spillway. A Jn value of 3.34 is taken based on 3 joint sets plus random jointing.

The Kb value for moderately hard rock is 50/3.34 = 15

Kb for very hard rock with very little fracturing is based on the case of intact rock with no or few joints and an estimated RQD value of 95. A Jn value of 1.0 is taken due to the massive nature of the rock.

The Kb value for very hard rock is 95/1 = 95.

Discontinuity/Interparticle Bond Shear Strength Number Kd

The discontinuity/interparticle bond shear strength number Kd is represented as Kd = Jr/Ja

Where: Jr = joint roughness number and Ja = joint alteration number.

For moderately hard rock, and based on field observations and joint separation criteria in the Guide, the joints are tight. Accordingly, the joint roughness condition is taken as rough/irregular, undulating (e.g., tension joints, rough sheeting joints, rough bedding). Based on these criteria, the joint roughness number Jr for moderately hard rock is 3.0.

For moderately hard rock and based on field observations and gouge condition criteria in the Guide, the field identification of gouge is "joint tightly healed with hard, nonsoftening, impermeable mineral filling, e.g. quartz, calcite, or epidote. Based on an aperture width (joint width) of less than 1.0 mm, the joint alteration number Ja for moderately hard rock is 0.75.

For moderately hard rock and the Jr and Ja numbers, the Kd number is 4.0

For very hard rock and based on field observations and joint separation criteria, Ja is taken at 0.75 for a tightly healed joint condition. The value Jr is taken at 4.0 for discontinuous joints.

For very hard rock and the Jr and Ja numbers, the Kd number is 5.3.

Relative Ground Structure Number Js

The relative ground structure number represents the orientation of the effective dip of the least favorable discontinuity with respect to spillway flow. This number accounts for effect of relative shape of material units or the ease at which spillway flow penetrates the ground and dislodges individual material particles.

For moderately hard rock and based on field observations, the value of Js taken at 0.50, to represent a range of values for joint dip directions with and against spillway flow, for dip angles ranging from 80 degrees to 10 degrees.

For very hard rock and based on field observations, the value of Js is taken at 1.00 for massive rock conditions.

Headcut Erodibility Index Kh

Based on the above estimated parameters, the headcut erodibility index Kh is calculated below for assumed uniaxial compressive strength values of 12.5 MPa for moderately hard rock and a relatively low value of 50 MPa for very hard rock (compressive strength range is 100 to more than 250 MPa).

Moderately Hard Rock: Kh for Ms of 12.5 MPa = (12.5)(15)(4)(0.5) = 375

Very Hard Rock: Kh for Ms of 50 MPa = (50)(95)(5.3)(1) = 25,175

Stream Power

For the PMF spillway inflow rate of 11,900 cfs, the flow depth H is taken at 5 feet, flow velocity V is taken at 35 feet per second and hydraulic slope S is taken at 0.20. In metric units, the Stream Power is calculated below.

Stream Power = (W)(V)(H)(S) = (999.6)(10.6)(1.5)(0.2) = 3,178.7

Erosion Potential - PMF Spillway Flow Conditions

Combining the erodibility index Kh with estimated stream power provides in indication of erosion potential for the rock exposed in the spillway and which could be used for a spillway subgrade.

For moderately hard rock, erodibility index of 375 and stream power of 3,178.7, the likelihood of erosion is high (erodibility threshold probability graph of Figure 7). Scour will occur in the spillway rock under the PMF flow conditions.

For very hard rock, erodibility index Kh of 25,175 and stream power of 3,178.7, the likelihood of erosion is low (erodibility threshold probability graph of Figure 7). Scour is not likely to occur in the very hard rock areas of the spillway.

Erosion Potential – 500 Year Inflow Event

For the estimated 500-year spillway inflow rate of 2,380 cfs, the flow depth H is taken at 1.5 feet, flow velocity V is taken at 12 feet per second and hydraulic slope S is taken at 0.20. In metric units, the Stream Power is calculated below.

Stream Power = (W)(V)(H)(S) = (999.6)(3.6)(.45)(0.2) = 323.9

For moderately hard rock, erodibility index of 375 and stream power of 323.9, the likelihood of erosion is high. Scour is likely to occur in the zones of moderately hard rock.

Since erosion of the moderately hard rock is likely to occur, provisions are necessary in the spillway to reduce potential for erosion. Hard to very hard rock exposed in the spillway channel area is resistant to erosion under both the 500-year and PMF inflow events and erosion protection provisions for hard to very hard rock is not required.

EXISTING DAM STRUCTURE

Structure Description

Geometry

The existing dam was designed and constructed as a homogeneous engineered fill structure. The dam is approximately 80 feet high at the maximum section with a crest length of approximately 435 feet. The top width is approximately 30 feet with an upstream embankment slope inclination of 3 horizontal to 1 and downstream slope inclined at 2 to 1. A cross-section illustrating the configuration of the existing dam is shown on Figure 8.

Foundation and Abutments

A 40-feet wide core trench, centered along the dam crest centerline, was excavated to clean, very hard, competent andesite bedrock. Alluvial materials of clayey gravels to gravelly, clayey sands were excavated to a depth of approximately 7 to 10 feet to expose the bedrock. The exposed bedrock surface was cleaned of soil and loose debris. Grout was hand-broomed into open fractures (joints) after they were cleaned. The dam foundation materials outside the core trench consist primarily of medium dense (firm) gravelly clay mixtures with lenses of sand. Some local deposits of soft to medium stiff gravelly clay also occur. Groundwater was encountered at depths of 8 to 10 feet below the ground surface in exploratory trenches upstream of the dam site and at depths of approximately 2 feet at the dam site. These trenches were excavated during the 1982 site geotechnical investigations.

The dam embankment was keyed into abutment materials of very hard andesite on the north end and andesitic breccia on the south end. The olivine andesite in the north abutment is described as visually fresh with minimal alteration along fractures, with latent to two-dimensional planes of separation. The dominant fracture or joint plane strikes North 80 degrees and dips 40 to 50 degrees south. Fracture frequency is on the order of 1 to 3 inches. The intensity of open fractures decreases toward the interior of the rock mass.

The volcanic breccia in the south abutment is described as visually fresh, massive, with a few randomly oriented fractures exposed in outcrops. The rock is described as a dacite with angular gravel to boulder-size clasts of varying volcanic composition.

Dam Embankment Fill Materials

Materials comprising the dam embankment fill were obtained from borrow areas in the Currant Creek floodplain and from an existing, breached dam that was located a short distance upstream of the new dam. The floodplain sources are alluvial deposits with typical variation in bedding due to alluvial depositional and erosional processes.

The presence of well-graded clayey to clayey silty, sandy gravels and clayey sands with interbedded, discontinuous sand, silt and clay lenses was thought to create handling, mixing and preparatory problems for construction of a zoned dam embankment. Accordingly, the dam was built as a homogeneous earth fill structure.

Field observations during the geotechnical investigations and results of laboratory testing on representative soil samples for the original dam design indicated that the predominant soil types are gravels with a sand-silt-clay matrix. These materials are classified as silty and clayey gravels (GM, GC) based on the Unified Soil Classification System (USCS). Soils classified as silts and clays (ML, CL) commonly contained gravel. Atterberg Limit tests indicated tested soils from the site are either non-plastic or have low to moderate plasticity.

Comparison of various soil properties determined from laboratory testing led to determination of a well-graded gravel-sand-silt-clay mixture as the representative soil for construction of the dam embankment. The maximum dry density for this material sample ranged from 110 to 115 pounds per cubic foot at an optimum moisture content of 14 percent, with a Plasticity Index less than 10. Permeability of this soil was estimated to be 0.0005 feet per day (Freeze and Cherry, 1979).

Toe Blanket Drain

Initial design planning for the dam included provision of an internal chimney drain to be located in the downstream part of the dam embankment. The chimney drain concept was abandoned and a horizontal blanket drain was installed beneath the toe of the dam embankment. The blanket drain extends from the embankment toe approximately 85 feet into the dam. The drain includes a 5-feet thick drain blanket sandwiched between two-stage filter layers on the top and bottom of the drain material. The approximate configuration of the toe drain is illustrated on Figure 8.

The drain blanket is 5 feet thick and consists gravel-sand in size ranges of 85 percent passing ¾”-inch, 50 percent passing ¼”- inch and 10 percent passing the #10 sieve. The first stage filter next to the drain blanket is 1 foot thick and consists of sand with minor gravel, with 90 percent passing the #4 sieve, 50 percent passing the #10 sieve and 25 percent passing the #16 sieve. The second stage filter, outside the first stage, is 1 foot thick and consists of sand in size ranges of 90 percent passing the #10 sieve, 50 percent passing the #50 sieve and 5 percent passing the #200 sieve. The total thickness of the drain and filters is 9 feet.

Three PVC drain pipes were installed to convey water out of the toe drain. Some minor water discharge is reported by YLC personnel during the spring season. No discharge is observed during the summer or winter months.

Outlet Pipe & Emergency Spillway

Water can be released from the reservoir through a 30-inch steel pipe encased on reinforced concrete. The outlet pipe is located near the north end of the dam. Water release is controlled by a hand-operated wheel valve on top of the dam near its north end. Water discharged through the outlet pipe flows into Currant Creek. A trash rack was installed at the pipe inlet. The emergency spillway was constructed near the south end of the dam.

Dam Embankment Protection

Riprap consisting of angular andesitic rock in approximate sizing of 2 inches to 12 inches was placed on the upper part of the upstream embankment slope. Wave action has created some shallow benching in the upstream slope. The downstream embankment slope supports grasses and some sage brush.

Flow Diversion during Dam Construction

Prior to dam construction, a flow diversion system was constructed to carry flows of Currant Creek through the dam site. This system consisted of a 48-inch diameter corrugated steel pipe extending through the dam site, adjacent to the south abutment. An earth fill cofferdam was constructed upstream of the dam site to contain flows and direct them to the bypass pipe. When the reservoir outlet pipe was installed near the north end of the dam, creek flows were directed through it and the bypass pipe was removed.

BASIS FOR ORIGINAL DESIGN OF EXISTING DAM AND SPILLWAY

Geotechnical & Engineering Geology Investigations

The basis for original design of the dam included extensive subsurface geotechnical investigations and geologic mapping. Investigation and mapping work were conducted to determine foundation conditions for the dam, geologic materials in the spillway area, borrow sites for dam embankment and riprap materials, and to evaluate stability of terrain adjacent to the proposed reservoir that might be effected by reservoir water and that might be unstable in proposed spillway channel cut slopes.

The investigations included 8 exploratory borings, 25 exploratory backhoe pits, 6 refraction seismic surveys and laboratory testing of representative soil samples. Four of the borings upstream of the dam were drilled with a Chicago Pneumatic 650 WS machine (air rotary). The remaining 4 borings in the dam footprint area were drilled with a Longyear core drilling machine, which enabled collection of core samples from penetrated bedrock beneath alluvial materials. Approximate locations of the borings, backhoe pits and seismic profiles are shown on Figure 9.

Laboratory testing included determination of grain size distribution of soil materials for dam construction, maximum dry density/optimum moisture content determinations of proposed fill material and triaxial shear testing. Laboratory tests were done by the certified CWEC materials testing laboratory. Results of the CWEC laboratory tests are shown on Table 1 (Appendix D). The triaxial shear testing was done by the CH2M testing laboratory in Portland, Oregon. This test, under undrained conditions resulted in a cohesion value of 1,800 psf and internal friction angle of 12 degrees.

Soil testing was conducted to determine density ranges for chunk samples of soil from backhoe pits, compacted samples for engineered fill, permeability assessments, strength parameters and suitability for use in the dam embankment. The triaxial shear testing was done under undrained conditions for stability analysis of the dam embankment slopes under rapid drawdown conditions.

Stability Analysis – Dam Embankment

Stability of the dam embankment was evaluated during the 1982 investigations by hand according to the "modified Swedish slip circle method. This method is based on division of the embankment above a selected sliding failure surface into slices and computing driving and resisting forces based on soil density and strength parameters of internal friction angle and cohesion.

The soil strength parameters were determined by triaxial shear testing under undrained conditions of an embankment soil sample remolded to 90 percent of the maximum dry density. The resulting angle of internal friction was 12 degrees and cohesion was 1,800 pounds per square foot. The values used for stability analysis were reduced to 6 degrees and 1,200 psf, respectively, for a measure of conservatism. The upstream slope angle for the analysis was 3 to 1 and the downstream slope angle was 2 to 1. Several trial analyses were conducted with various failure surface locations and configurations, until the lowest safety factor against sliding was obtained. This value was 1.6 for the downstream slope and for the upstream slope, which was under undrained conditions.

SEISMIC ANALYSIS

Purpose

The Currant Creek Dam is classified by the OWRD-DSS as a high hazard structure. Accordingly, an analysis of the dam is required to account for seismic loading of the dam embankment and related potential for deformation and related loss of freeboard at the dam or failure of the embankment slopes. Seismic loading must be determined for a Maximum Credible Earthquake (MCE). An analysis of potential for liquefaction due to a seismic event is also required. The purpose of the seismic analysis is to fulfill this requirement.

Potential for Deformation or Failure of Dam Embankment Slopes

Approach

The seismic analysis of Currant Creek dam included estimation of the peak ground acceleration at the dam site and related potential for deformation of the dam due to an MCE. The yield coefficient (Ky) was also determined and used to estimate deformation potential and probability of deformation occurrences.

The analysis was conducted according to methods reported in the following papers:

1. A.G. Hull & A. Augello, and R.S. Yeats, 2003. Deterministic Seismic Hazard Analysis in Northwest Oregon, U.S.A., 2003 Pacific Conference on Earthquake Engineering.

2. Bray, Jonathon D., Simplified Seismic Slope Displacement Procedures, 2007. Earthquake Geotechnical Engineering, 327-353.

3. Bray, Jonathan D., Travasarou, Thaleia, 2011. Pseudostatic Slope Stability Procedure, 5th International Conference on Earthquake Geotechnical Engineering, Paper No. Theme Lecture 1.

4. Abrahamson, N.A. & Silva, W.J., 1997. Empirical Response Spectral Attenuation Relations for Shallow Crustal Earthquakes, Seismological Research Letters, Vol 68 (1). 94-127.

5. Sadigh, K.; Chang, C.Y.; Egan, J.A., Makdisi, F.; Youngs, R.R. 1997. Attenuation Relationships for Shallow Crustal Earthquakes Based on California Strong Motion Data, Seismological Research Letters, 68 (1). 190-198.

6. Hanks, T.C. & Bakun, W.K. 2002. A Bilinear Source-Scaling Model for M-log A Observations of Continental Earthquakes, Bulletin of the Seismological Society of America 92 (5). 1841-1846.

The above-cited paper by Hull and Augello evaluates 54 possible earthquake sources to determine controlling MCE within 200 km of 12 Portland General Electric dam sites in northwest Oregon. Accordingly, this work is considered useful for the Currant Creek dam analysis and was utilized in this analysis. This report is included in Appendix G.

The analysis approach included the following steps:

Step 1: Determining earthquake sources;
Step 2: Determine locations of nearest mapped faults to the dam location;
Step 3: Determine Potential Site Response to Earthquake;
Step 4: Determine parameters and yield coefficient for seismic deformation analysis;
Step 5: Cross-check yield coefficient with a range of calculation methods.
Step 6: Conduct Seismic Deformation Analysis.

Analysis Step 1 - Earthquake Sources

The first step in the seismic analysis was determination of potential earthquake sources. Earthquake sources are described in the "Earthquake Hazards Maps for Oregon," 1996, Oregon Department of Geology and Mineral Industries (seismic design mapping by Geomatrix Consultants, Inc. for the Oregon Department of Transportation). Sources of earthquakes include 1) crustal earthquakes; 2) intraplate or Wadati-Benioff earthquakes; and 3) great subduction earthquakes.

Crustal earthquakes are the most common and occur on relatively shallow faults (within 10 miles of the ground surface). According to seismic literature, they occur on relatively short and shallow faults, which are not always visible at the ground surface. The Magnitude for these events might reach 6, based on the existing available and relatively short record. Literature states that many faults in Oregon are believed by geoscientists to be capable of Magnitudes of 6.5 to 7.0. These earthquakes are relatively common in the Portland area and northern Willamette Valley, off the southern Oregon coast, northeastern Oregon and in areas of southeastern Oregon. Earthquake shaking in most areas east of the Cascade Mountains is generated by crustal faults.

Intraplate or Wadati-Benioff earthquakes occur within the remains of ocean floor that has been pushed through subduction, beneath North America. Ground shaking from these events occurred in the Puget Sound in 1949 and 1965, and Portland in 1949. This type of earthquake is believed possible for occurrence with Magnitude of 7.0 to 7.5 anywhere beneath the Oregon coast range or western Willamette Valley.

Great subduction earthquakes occur around the world in subduction zones where large segments of the earth's crust are pushed very deep into the earth. These earthquakes are among the more powerful recorded events with Magnitude of 8.0 to 9.0. The Cascadia Subduction Zone off the Oregon-Washington coast is an example. Subduction earthquakes on this Zone appear to occur on average every 350 to 500 years. The last know subduction earthquake on this Zone occurred on January 26, 1700.

Analysis Step 2 – Determine Mapped Locations of Nearest Faults to Currant Creek Dam

Site Location Coordinates

The existing Currant Creek Dam is located in Wasco County, in the Southeast ¼ of the Southeast ¼ of Section 35, Township 8 South, Range 18 East, Willamette Meridian. The site Latitude and Longitude coordinates are 44.823 degrees and 120.528 degrees, respectively.

Fault Locations – "Geologic Map of Oregon"

The "Geologic Map of Oregon," by Walker and MacLeod, 1991, shows locations of mapped and inferred faults. A mapped fault is shown over a length of about 6 miles, passing through the approximate reservoir area. The fault trend is northeast to southwest. No age is assigned to the fault.

A fault trace with a north-south trend over a length of about 6 miles is mapped about 8 miles west of the site. No age is assigned to the fault. The fault is not shown on other maps of earthquake faults.

Fault Locations – "Map of Oregon Earthquake Faults"

The site location relative to locations of mapped faults was estimated through use of the "Map of Oregon Earthquake Faults," prepared by the Oregon Department of Geology and Mineral Industries, "Earthquake Hazard Maps for Oregon," 1996. The map shows locations of faults considered "Recent" in age. Figure 10 shows the approximate dam site location relative to the approximate location of mapped faults within a 75-mile radius of the site. Approximate distances between the dam location and the faults are summarized below for Holocene, Late Quaternary and Quaternary-age faults. The locations are rough considering the detail level of the map and no map scale.

Holocene – Late Pleistocene (movement in last 20,000 years)
The locations of mapped faults nearest to the site are listed below:

1. Approximately 75 miles west-northwest of the site in eastern Clackamas County (Mt Hood area);
2. Roughly 75 miles southwest in Deschutes County, near Sunriver;
3. Roughly 60 miles southwest in Deschutes County (Sisters Fault Zone);
4. Roughly 65 miles north, near the Columbia River.

Late Quaternary (movement in last 780,000 years)
The mapped faults nearest to the site are listed below:

1. Roughly 45 miles west, between Breiten Bush Hot Springs and Warm Springs;
2. Roughly 60 miles southwest, in Deschutes County; northwest-trending fault zone (Sisters Fault Zone);
3. Roughly 50 miles south, near Post, Oregon; and
4. Roughly 60 miles north, near Arlington.

Quaternary (movement in last 1,600,000 years)
The mapped faults nearest to the site are listed below:

1. Roughly 25 miles northeast, southeast of Condon, Oregon;
2. Roughly 30 miles north, west-northwest of Condon, Oregon;
3. Roughly 40 miles northwest, north of Maupin;
4. Roughly 60 miles west-southwest, high Cascade Mountains area; and
5. Roughly 70 miles south-southwest, in Deschutes County, west of Brothers, Oregon.

Fault Locations – "Map of Selected Earthquakes for Oregon, 1841 through 2002"

Epicenters of historic earthquakes in Oregon are shown on the map "Map of Selected Earthquakes for Oregon, 1841 through 2002," DOGAMI, Open-File Report 03-02. This map is shown on Figure 11. Review of the map shows epicenters for 27 earthquakes with a Magnitude range of 5.0 to 5.9. Most of these are in an area between Salem and Portland, and the coast. One epicenter is mapped for a Magnitude range of 6.0 to 6.9 near Klamath Falls. The remaining mapped epicenters are for events with Magnitude ranging from 0 to 4.9. Clusters of these relatively small events are mapped in areas of Portland, an area northeast of Salem, Mount Hood about 65 miles northwest of the dam, a Wasco County area about 25 miles northwest of the dam, the Condon area about 35 miles northeast of the dam, and Klamath Falls area. A low-density scatter of epicenters is mapped in the approximate area between Condon and the Oregon border northeast of Pendleton.

The historic record indicates that earthquakes in Oregon are generally associated with four geographic areas of notable seismic activity. These include the Cascade Seismicity Zone, Portland Hills Seismicity Zone, South-Central Oregon Seismicity Zone and the Northeastern Oregon Seismicity Zone. These zones are described on Figure 11.

The Portland Hills and Cascade Seismicity Zones (areas of Portland northeast of Salem) are the nearest source Zones to the dam and reservoir site for historic events in the larger Magnitude range of 5.0 to 5.9. The approximate distance between the site and mapped epicenter locations for these events is about 100 miles.

Figure 12 "Map of Oregon Earthquake Faults" shows mapped locations of Holocene and Late Quaternary faults. The nearest mapped Holocene fault is on the northwest side of Mount Hood, about 70 miles distant. The nearest mapped Late Quaternary fault is about 40 miles west of the dam. Faults at this location are consistent with those mapped on Figure 10.

Fault Locations – Recently Discovered Mount Hood Faults

A network of faults was discovered by staff (Ian Madin, William Burns, Lina Ma, Ashley Streig) of the Oregon Department of Geology and Mineral Industries (DOGAMI) in 2018 that are believed capable of generating a Magnitude 7.2 earthquake. These faults were described as potential sources for a crustal earthquake. As reported in 2018, little is known of slip rates, history or recurrence intervals. The network includes four distinct fault segments spanning 34 miles from Clear Lake, south of Mount Hood, north to the Columbia River. The longest fault spans 3 miles. It was reported that "If any fault sections ruptured independently of each other, it could trigger a Magnitude 7.7 earthquake." This fault network is located about 65 miles northwest of the dam and reservoir site.

Site Location Relative to Cascadia Subduction Zone Fault

The approximate location of the epicenter for the Cascadia Subduction Zone Fault is shown on Figure 13 at a location off the Oregon coast about 258 miles from the Currant Creek dam site. This is a planning scenario map for a Magnitude 9.0 event. For this scenario, the estimated intensity and potential damage are also shown in the table beneath the map. The planning scenario shows perceived shaking and potential damage estimated at weak to light and no damage potential, respectively.

Analysis Step 3 – Determine Possible Source & Magnitude; Maximum Credible Earthquake

Possible MCE Source Fault

Considering the availability of earthquake data as described above, a spectrum of earthquake events exists from which a reasonable MCE could be estimated.

The recently discovered (DOGAMI, 2018) faults at Mount Hood are about 68 miles (109 km) northwest of the dam and in very close proximity to a fault with movement in the last 20,000 years shown on Figure __, "Map of Selected Earthquakes for Oregon." DOGAMI staff indicate this new-discovered fault network is capable of a Magnitude 7.2 event. If any fault segment ruptured independently of the others in the network, the resulting Magnitude could be 7.7.

The nearest mapped, potentially active fault traces to the dam and reservoir site are roughly 75 km (47 miles) to the west and 61 km (38 miles) to the northwest. The fault network to the west is Late Quaternary in age and reported as active with movement occurring between 20,000 and 780,000 years ago as shown on Figure 11, "Map of Selected Earthquakes for Oregon". This fault system is consistent with the location of the Warm Springs Fault, identified as a possible MCE source for the Pelton Re-regulating dam, Pelton Arch dam and Round Butte Dam as described in the Hull, Augello and Yeats paper referenced above. Accordingly, this system is considered as a possible source of an MCE which could affect the Currant Creek dam.

The fault mapped roughly 61 km northwest of the dam is shown as a thrust fault. It is not shown on the "Map of Selected Earthquakes for Oregon" (Figure 11). It is shown on the Hull, et al report map and on the "Geologic Map of Oregon", Walker and MacLeod, 1991, USGS. This fault is also considered a potential MCE source.

Possible Magnitude Estimate

Following the procedures described in the above-referenced Hanks and Bakun paper, the Magnitude of the possible source MCE was estimated on the basis of fault rupture length and width for the Warm Springs Fault. The Hull, Augello and Yeats paper report a rupture length of 30 km for the fault. The width of the fault is taken at 15 km, consistent with the paper. Based on the rupture length and width, the fault area is 450 square km. Correlations between fault area and Magnitude reported by Hanks and Bakun indicate that the Magnitude of the possible source MCE on the Warm Springs Fault could be 6.7. The mapped trace of the thrust fault is similar in length to the Warm Springs Fault as can be estimated at the rough scale of the Hull, et al map. It is assumed that the potential Magnitude range for this fault is generally similary to that of the Warm Springs Fault. For this report, a Magnitude of 7.0 was selected for seismic analysis.

Analysis Step 4 – Determine Possible Peak Ground Acceleration

Peak Ground Acceleration -Deterministic Estimate

The peak ground acceleration (PGA) for the Pelton Re-regulating dam from a Magnitude 6.7 MCE on the Warm Springs Fault was reported (Hull, et al) as 0.22g. This dam is roughly 21 km from the fault. This value was cross-checked with correlations between PGA, earthquake Magnitude, distance from the fault and strike-slip fault conditions developed by Abrahamson, et al. For a distance of 21 km between the re-regulating dam and the fault, and Magnitude range of 6.8 to 7.2, the PGA at the Pelton Re-regulating dam would be about 0.22g, consistent with the peak ground acceleration given by Hull, et al for the dam site.

The Currant Creek dam is located roughly 75 km northeast from the Warm Springs Fault based on the map of Hull, et al. Abrahamson, et al data for a strike-slip fault and rock conditions indicates the PGA at the dam site for an earthquake Magnitude range of 6.8 to 7.2 could be 0.04g, reflecting attenuation effects with distance from the fault.

Consideration was also given to PGA at the dam site resulting from a MCE on the recently discovered Mount Hood fault system that is considered active, with potential for a Magnitude 7.7 (reported by DOGAMI staff as possible). The Currant Creek dam is located roughly 109 km from this fault system. At this distance and the highest Magnitude range on the correlation graph (Abrahamson, et al) of 7.3 to 7.5, the PGA at the Currant Creek dam would be about 0.045g for strike-slip fault and rock conditions.

The location of the dam relative to a historic earthquake epicenter shown on the Hull, et al fault map was considered. The mapped epicenter is for a Magnitude 4.5 to 5.4 event. Currant Creek dam is located roughly 34 km (21 miles) southeast of the epicenter (located southeast of Maupin). Abrahamson, et al data shows a potential PGA value of 0.11g at the dam site for a Magnitude 6.8 to 7.2 event at the epicenter site, under strike-slip and rock conditions.

Finally, consideration was given to the location of the dam relative to a potentially active fault shown on the Hull, et al map as a thrust fault. The dam is located roughly 61 km southeast of the fault. Abrahamson, et al data shows a potential PGA of 0.045g for a Magnitude 6.8 to 7.0 event based on strike-slip fault conditions. It is possible, although not certain, that a thrust fault could generate a higher Magnitude and PGA value. Abrahamson, et al data for a Magnitude range of 7.3 to 7.5 was considered and found to result in a potential PGA at the dam site of just under 0.10g.

Conclusion – PGA at the Dam Site

The faults included in the analysis of potential PGA at the Currant Creek dam site are in relatively near proximity to the dam and representative of potential sources of a MCE which could impact the dam. Analysis based on the Hull, et al and Abrahamson, et al data indicates that PGA at the dam site is in the range of 0.045 to 0.11g. For purposes of this report, a PGA value of 0.11 is assumed.

Peak Ground Acceleration – Probabilistic Estimate

Potential degree of ground shaking and peak ground acceleration for the dam site area was further evaluated through review of "Probabilistic Hazard Maps" which are included in the above-cited (Step 1) "Earthquake Hazard Maps for Oregon." These maps were also prepared by Geomatrix Consultants, Inc. to provide estimates of the likely strength of future earthquake shaking throughout Oregon.

The probabilistic hazard maps are for the 500-year, 1000-year and 2,500-year recurrence intervals. The time window for analysis use depends on the nature of the project. The 2,500-year map would be used for extremely important or hazardous facilities. For facilities with little risk of death or injury, the 500-year map could be appropriate. The 1000-year map could be used for other applications involving life safety. Findings from review of the three maps are summarized below.

Review of 500-Year Shaking Map

This map on Figure 14, shows maximum earthquake shaking (Peak Ground Acceleration) expected in Oregon with a frequency of occurrence of once in 500 years (10% chance of occurrence in any 50-year period). The map shows that the dam site is located in a zone shown to have a peak ground acceleration of 6 percent of gravity (0.06g). In this zone, shaking will be felt by most and objects move.

Review of 1000-Year Shaking Map

This map on Figure 15, shows maximum earthquake shaking expected in Oregon with a frequency of occurrence of once in 1000 years (5% chance of occurrence in any 50-year period). The map shows that the dam site is located in a zone shown to have a peak ground acceleration of 8 percent of gravity (0.08g). In this zone, shaking will be felt by all, heavy objects will move and damage will be slight.

Review of 2,500-Year Shaking Map

This map on Figure 16**,** shows maximum earthquake shaking expected in Oregon with a frequency of occurrence of once in 2,500 years (2% chance of occurrence in any 50-year period). The map shows that the dam site is located in a zone shown to have a peak ground acceleration of 10 percent of gravity (0.10g). In this zone, shaking will be felt by all, heavy objects move and damage will be slight.

Summary -Estimated Peak Ground Acceleration

In summary, the estimated peak ground acceleration at the dam site from the assumed MCE on the Warm Springs Fault and estimated ranges of peak ground acceleration based on probabilistic hazard maps ("Probabilistic Hazard Maps" included in "Earthquake Hazard Maps for Oregon," prepared by Geomatrix Consultants, Inc.) show that the range of peak ground acceleration for the dam site area is relatively low, between 0.05 and 0.10g. This range is consistent with the deterministic range of 0.045 to 0.11 determined in the preceding section of this report. Probabilistic data show that shaking will be felt by most to all people and damage ranges from none to slight.

Analysis Step 5 – Determine Affects of Possible MCE on Currant Creek Dam

The potential effect of an MCE on the dam is a function of earthquake Magnitude, length of possible source fault rupture and width of fault, distance of possible source fault from the dam, potential for material strength loss in the dam and foundation, spectral acceleration at the degraded period of the dam structure, and yield coefficient Ky. Different methods exist for determining potential effects of an MCE on dams.

Considered Methods of Analysis – Potential Affects of MCE on Currant Creek Dam

Three simplified methods were considered for determining potential for seismic deformation of the Currant Creel dam. These include 1) N. M. Newmark, "Effects of Earthquakes on Dams and Embankments," (Fifth Rankine Lecture). 1965; 2) Jonathan D. Bray, "Simplified Seismic Slope Displacement Procedures," Department of Civil and Environmental Engineering, University of California, Berkeley, USA, 2007; Jonathan Bray and Thaleia Travasarou, "Pseudostatic Slope Stability Procedure," 2011; and 3) Chi-Chin Tsa and Yu-Chun Chien, Assistant Professor, Department of Civil Engineering and Graduate Research Assistant, respectively, both of the National Chung Hsing University, Taichung, Taiwan.

All three methods involve calculation of the yield coefficient Ky, which is used to determine the probability of negligible displacement and the amount of deformation. The Ky value is critical to this probability determination. The three methods were used primarily for Ky value determination. Bray et al, had evaluated other methods, including the Newmark method, and developed an upgraded approach for determining probability of displacement and amount of deformation in embankments. Work of Bray and Travasarou (2011) also includes development of spreadsheet tools for estimating yield coefficient Ky and slope displacements in dams which are included in Appendix E of this report.

The above-cited Bray publication states that seismically induced permanent displacements are typically calculated with the Newmark sliding block analogy. The Bray publication critiques some commonly used procedures and recommends use of the simplified procedure described in the Bray publication for use in engineering practice. The method described by Bray was selected for analysis of potential deformation of Currant Creek dam due to an MCE event.

Components of Seismic Displacement Analysis - Bray

Bray, et al describes three critical components of a seismic displacement analysis. These include: (1) earthquake ground motion; (2) dynamic resistance of the structure; and (3) dynamic response of the potential sliding mass. These components are described below.

Earthquake Ground Motion
Bray, et al reports that use of peak ground acceleration (PGA) alone is over-simplistic since ground motions with identical PGA values can vary significantly relative to frequency and duration, and in effects to slope stability. Characterization of equivalent seismic loading on a structure from earthquake ground motion has been commonly done in earthquake engineering by use of spectral acceleration. Bray reports that use of the 5 percent damped elastic spectral acceleration at the degraded fundamental period of the potential sliding mass is the optimal ground motion intensity measure relative to efficiency and sufficiency. In this regard, the damped spectral acceleration minimizes variability in its correlation with seismic displacement and makes the relationship independent of other variables.

Bray, et al state that no single period-independent ground motion parameter has been found to be adequately efficient for slopes of all dynamic stiffnesses and strengths. However, the spectral acceleration at a degraded period of the potential sliding mass equal to 1.5 times the initial fundamental period of the slope is the most efficient ground motion parameter for all slopes. This approach requires estimation of the fundamental period of the potential sliding mass based on height of the mass and shear wave velocity of the mass.

Dynamic Resistance
The overall dynamic resistance of an earth structure is represented by its yield coefficient (Ky). The Ky value depends primarily on the dynamic strength of the material along the critical sliding surface, geometry and weight of the earth mass. The Ky value has always been used in simplified seismic analyses with sliding block or circular sliding-surface procedures. Its effect on seismic displacement makes it a critical parameter for analysis.

The Ky parameter can be determined from the Newmark and the Bray methods. It can also be determined by an updated method proposed more recently by Chi-Chin Tsa and Yu-Chun Chien, Assistant Professor, Department of Civil Engineering and Graduate Research Assistant, respectively, both of the National Chung Hsing University, Taichung, Taiwan. This method of Ky calculation is based on their experience with earthquakes and slope failures in Taiwan. Ky values for the Currant Creek dam were determined according to all three methods.

Dynamic Response

Bray, et al reports that seismic displacement also depends on the dynamic response characteristics of the potential sliding mass. Holding all other factors constant, Bray states that seismic displacements increase when the sliding mass is near resonance compared to that calculated for very stiff or very flexible slopes. Bray reports that the Newmark rigid sliding block model ignores the dynamic response of a deformable sliding mass, whereas the Bray method provides for dynamic response through use of the initial fundamental period (Ts) of the structure in the analysis. This value is estimated by the equation Ts = 4H/V, where H is the average height of the potential sliding mass and V is the average shear wave velocity of the potential sliding mass. The equation applies to a relatively wide sliding mass shaped either like a trapezoid or segment of a circle. The Ts value is estimated by the equation Ts = 2.6H/V for a triangular-shaped sliding mass.

Justification - Bray Method

The yield coefficient (Ky) represents dynamic resistance of slopes under earthquake loading conditions and is critical in predicting seismic probability and amount of displacements in earth materials. The Ky value for a given embankment material is the horizontal seismic force in percent gravity that will reduce the safety factor found in a pseudostatic analysis to 1.0.

Newmark refers to an "N" value, corresponding to the Ky value described by Bray and Chi-Chin Tsa and Yu-Chun Chien. Calculation of the N or Ky values by Newmark and Bray methods are done by iterative trial and error calculations based on various combinations of block, wedge or circular failure modes, soil cohesion and soil friction angle. The proposed Chi-Chin Tsa and Yu-Chun Chien method was developed considering the need for laborious iterative calculations by the other methods. Their work resulted in a relatively simple and expedient method to calculate Ky. Bray and Travasarou (2011) describe a process to calculate Ky based on selection of allowable deformation, spectral acceleration at the degraded period of the structure, initial fundamental period of the structure and Magnitude.

The method proposed by Bray and Travasarou (2011) was developed after their review and critique of other simplified seismic displacement methods for earth structures. Uncertainty in assessing the likely performance of an earth structure during an earthquake is due primarily to the input ground motion. Accordingly, development of the Bray method was based on results of sliding block analyses conducted with a database of "hundreds of recorded ground motions" that have become available for many earthquake events.

The Newmark sliding block method is described in the Bray publication to capture the part of seismically induced permanent displacement attributed to deviatoric shear deformation. This means either rigid body slippage along a distinct failure surface, or distributed deviatoric shearing within the deformable sliding mass. The Bray method was developed to account for mechanisms consistent with the Newmark method, including deviatoric-induced displacement due to sliding on a distinct plane and distributed deviatoric shearing within the slide mass.

Estimation of earthquake-induced deviatoric deformations in earth structures calculated with the Bray method depends primarily on the spectral acceleration of the ground motion at the degraded period of the structure, the structure's yield coefficient and fundamental period. Bray describes the method as a seismic displacement model for estimating seismically-induced displacements that are generally consistent with documented cases of earth dam and solid-waste landfill performance. Bray also states that the method provides assessments that are not inconsistent with other simplified methods.

The Bray method provides assessments with an "improved characterization of the uncertainty involved in the estimate of seismic displacement" and can also be used for probabilistic and deterministic analyses. Bray states that estimations of seismic displacement using the method he describes should be considered "merely as an index" of expected seismic performance of an earth structure.

Analysis – Potential Effects of MCE Event on Currant Creek Dam

Bray and Travasarou (2011) describe a step-wise procedure for determining potential for deformation of a dam embankment. This approach was followed in analysis of deformation potential at Currant Creek dam based on an MCE event at the Warm Springs Fault. The analysis is described below.

Evaluate if Materials are Present that could lose Significant Strength from Earthquake Shaking
This is the most critical design issue (Bray & Travasarou, 2011). Materials reported as most stable are clays with low sensitivity, in a plastic state, dense sand either above or below the water table, and loose sand above the water table. The most sensitive materials include slightly cemented materials such as loess and submerged or partly submerged loose sand. The dam embankment fill and the foundation materials, other than rock beneath the core trench, consist of silt-clay-sand and gravel mixtures. The embankment materials were placed in lifts and compacted to a dense condition. Foundation materials are also in a relatively dense condition. These materials are not subject to severe strength loss as a result of earthquake shaking.

Establish the Allowable Seismic Displacement
H. Bolton Seed (1979) developed a pseudostatic slope stability method which is one of the commonly used procedures for earth dams with materials that do not undergo severe strength loss and crest acclerations less than 0.75g. It involves several simplifying assumptions wherein Seed assumed that a seismic displacement of one meter is acceptable performance (Seed, H.B., 1979, "Considerations in the Earthquake-Resistant Design of Earth and Rockfill Dams". Geotechnique, V.29(3), pp 215-263). Following Based on Bray and Travasarou (2011) allowable displacement was taken at 5 centimeters.

Determine the Initial Fundamental Period of the Potential Sliding Mass (of the dam)
For this case, the sliding mass is taken as a triangular-shaped mass which roughly approximates the shape of a slope failure mass in a dam embankment. For this case, the initial fundamental period Ts = 2.6H/Vs,

Where: Ts = initial fundamental period of the potential sliding mass;
H = average height of the potential sliding mass, in meters;
Vs = average shear wave velocity of the sliding mass.

The average height of the dam, after raising it 5 feet, is 29 meters. The shear wave velocity range for stiff soil, assumed for the compacted dam embankment, 600 to 1,200 feet/second. The average value of 900 feet/second, or 273 meters/second, was taken for calculation of the initial fundamental period of the sliding mass. Based on the average height and shear wave velocity, the initial fundamental period is 0.28 seconds.

Determine the Degraded Period of the Sliding Mass
Based on Bray and Travasarou (2011), the degraded period of the sliding mass is estimated as $T's = 1.5Ts$.

Where: Ts = initial fundamental period of the sliding mass.

Based on an initial fundamental period of 0.28 seconds, the degraded period of the sliding mass is 0.42 seconds.

Estimate the Spectral Acceleration at the Degraded Period of the Sliding Mass
Bray and Travasarou (2011) state that the best estimate of the spectral acceleration at the degraded period of the sliding mass can be made based on the mean of median predictions from multiple ground motion models. For this purpose, work by Abrahamson et al (2008) was used to estimate spectral acceleration at the degraded period. Based on the degraded period of 0.42 seconds, distance of 61 km between the dam and the thrust fault to the northwest and rock foundation conditions, the spectral acceleration is approximately 0.22g.

Spectral acceleration at the dam for the epicenter location at roughly 34 km from the dam was also determined. For this case, the spectral acceleration is estimated at 0.26.

Calculation of Ky
Following Bray and Travasarou (2011), the calculation is based on the selected allowable deformation of 5 centimeters, spectral acceleration of 0.22g, initial fundamental period of 0.28 seconds, degraded period of the structure of 0.42 seconds and Magnitude of 7.0. The calculation, shown on the Bray-Travasarou spreadsheet as Exhibit A in Appendix E, results in a Ky value of 0.07.
Ky was also estimated for the case of the epicenter for the historic earthquake located roughly 34 km from the dam site. For this calculation, the spectral acceleration value of 0.26 was used. The resulting Ky value is 0.085.

The Ky values of 0.07 and 0.085 were input into the calculation of potential displacement in the dam embankment. The spreadsheet for this calculation is shown as Exhibit B in Appendix E. The calculation results in estimated potential displacement in the dam for the various exceedance levels as shown below:

EXCEEDANCE LEVEL	DISPLACEMENT Ky = 0.07	DISPLACEMENT Ky = 0.085
16 Percent Exceedance:	2.4 centimeters	2.4 centimeters
50 Percent Exceedance:	4.8 centimeters	4.9 centimeters
84 Percent Exceedance:	9.5 centimeters	9.5 centimeters

If these Ky values are applied to the dam through a pseudostatic stability analysis and the safety factor is greater than one, the dam can be judged to perform satisfactorily. A pseudostatic stability analysis was performed on the dam to determine the Kv value which corresponds to a safety factor of 1.0. This is described below.

Pseudostatic Stability Analysis - Method

Technical literature (U.S. Bureau of Reclamation "Design Standards No. 13, Embankment Dams, Chapter 4: Static Stability Analysis Phase 4 (Final)", October 2011) on slope stability analysis states that the Spencer and Morgenstern-Price methods satisfy complete equilibrium of the sliding mass. The simplified Bishop method satisfies only vertical force and overall moment equilibrium, but not horizontal force equilibrium. The Janbu method satisfies only vertical and horizontal force equilibrium and not moment equilibrium. The Bishop and Janbu methods are relatively easy for stability analysis and typically provide conservative safety factor values when compared to the more accurate methods such as the Spencer and Morgenstern-Price methods.

The Spencer method is preferential according to the above cited USBR document. Preference is based on the limit equilibrium procedure of the method that: "1) assumes the resultant side forces acting on each slice are parallel to each other, 2) satisfies complete statics, and 3) is adapted for calculating factor of safety and side-force inclination for circular and non-circular shear surface geometries."

The stability analysis was conducted with STABL 6 software according to the Spencer method on the maximum cross-section of the existing dam and on the maximum cross-section of the dam after raising it 5 feet for the proposed increase in storage capacity. Analyses were done for Case I and Case II.

Case I analyses included four scenarios including existing and proposed dam condition. Case II included three scenarios based on rapid drawdown conditions for the existing dam and proposed dam raise.

The purpose of the stability analyses was to determine the factor of safety against embankment sliding for steady-state seepage and rapid drawdown conditions. A pseudostatic stability analysis was also performed to determine the yield coefficient for evaluating deformation potential for the dam under seismic conditions.

Stability Analysis Outputs

Each stability simulation generates a graphic display of the 10 most critical failure surfaces including the most critical failure surface. Output also includes a histogram showing the percent distribution of calculated factors of safety for all failure surfaces generated in the simulation. The histograms helped determine whether progressive simulations were improving the number of failure surfaces being generated close to the critical region. Improvement is evidenced by histograms with the largest percentages of failure surfaces with factors of safety closest to the minimum. This indicates a higher chance that the minimum safety factor for the simulation is in fact the minimum for the slope. Simulation results and related histograms are presented in Appendix E.

Analysis Parameters

The upstream embankment slope angle for analysis is 3 horizontal to 1 vertical and the downstream embankment slope angle is 2 to 1. The phreatic line through the dam was located according to a maximum normal operating elevation of the reservoir, extending through the dam to the toe drain. Analysis of the downstream embankment slope was based on steady-state seepage. Analysis of the upstream slope was based on soil saturation and effective stress conditions and also for rapid drawdown conditions. Soil strength parameters for analysis included the following:

Density and Shear Test Strength Parameters from 1982 Geotechnical Investigations

Embankment Soil

Unified Soil Classification: (based on lab tests) Clayey Gravel (GC)-Clayey Sand (SC)

Assumed representative maximum dry density (Modified Proctor)	115.0 pcf
Assumed optimum moisture content (Modified Proctor)	14.0 %
Wet density (based on 90% compaction per Modified Proctor):	118.0 pcf
Saturated density (below phreatic surface):	125.0 pcf
Cohesion (based on undrained triaxial shear test, original dam design):	1,800 psf
Internal friction angle (based on undrained triaxial shear test):	12.0 degrees

Foundation Soil (overlying bedrock)

Unified Soil Classification (based on lab tests)	Clayey, silty, sandy gravel (GM/GC)
Assumed representative saturated density (natural insitu):	119.0 pcf
Cohesion (estimated):	300.0 psf
Internal friction angle (estimated):	30 degrees

Bedrock

Rock classification:	Slightly weathered, very hard andesite
Estimated density (insitu):	145.0 pcf
Assumed cohesion:	0.0 psf
Assumed internal friction angle (estimated):	45 degrees

Of the 10-most critical failure surfaces determined by the analysis, the minimum safety factor for the upstream dam embankment slope under rapid drawdown conditions is 3.36. The minimum safety factor for the downstream slope under static, steady state seepage conditions is 1.83.

Adjusted Strength Parameters, Cohesion and Internal Friction

Stability analysis was also performed by assignment of cohesion and internal friction values that would provide a minimum safety factor at or very close to 1.5 for the downstream embankment slope of the dam. Discussion with OWRD-DS indicated that the desired safety factor against sliding of a dam embankment is 1.5. Accordingly, analysis was done to check reasonableness of soil strength parameters that would bring the safety factor to or very close to 1.5. This was also done as a measure of conservatism for estimation of the yield coefficient to be used in estimating probability that seismic deformation of the dam would be negligible as discussed below.

Adjustments of soil strength parameters cohesion and internal friction angle were made for the dam embankment fill material. The cohesion value of 1,800 psf was reduced to 800 psf. The internal friction angle was changed from 12 degrees to 30 degrees, assuming a more granular fill material.

Analysis Results

Case I

Four scenarios were subjected to slope stability analysis. The scenario description and analysis results are described below.

Scenario 1

Scenario 1 is the existing dam and maximum operating reservoir elevations of 1831 feet. Density and shear test strength parameters from 1982 geotechnical investigations described above were used in the Scenario 1 analysis. The safety factor against sliding for the upstream dam embankment slope under steady-state seepage and effective stress conditions is 3.16. The safety factor for the downstream slope under steady state seepage conditions is 1.83.

Scenario 2

Scenario 2 is the proposed dam raise by 5 feet to elevation 1850 feet and proposed maximum reservoir operating elevation raise to 1845 feet (inflow design flood elevation). Density and shear test strength parameters from 1982 geotechnical investigations described above were used in the Scenario 1 analysis. The safety factor against sliding for the upstream dam embankment slope under steady-state seepage and effective stress conditions is 3.49. The safety factor for the downstream slope under steady state seepage conditions is 1.56.

Scenario 3

Scenario 3 is the proposed dam and reservoir elevation raises with adjusted soil strength parameters to obtain a safety factor at or very near 1.5. Strength parameters were adjusted to a cohesion value of 800 psf and internal friction value of 30 degrees as describe above under "Adjusted Strength Parameters, Cohesion and Internal Friction." The safety factor against sliding for the upstream dam embankment slope under saturation and effective stress conditions is 3.10. The safety factor for the downstream slope under steady state seepage conditions is 1.51.

Scenario 4

Scenario 4 is a case of assumed liquefaction in the foundation soils beneath the lower part of the upstream dam embankment slope. Although liquefaction is unlikely based on analysis described in Analysis Step 6 - Liquefaction Analysis, described later in this report, this is based on a Cyclic Stress Ratio (CSR) of 0.49 determined for an element subjected to liquefaction analysis. Strength parameters were adjusted to a cohesion value of 800 psf and internal friction value of 30 degrees as describe above under "Adjusted Strength Parameters, Cohesion and Internal Friction." These parameters were taken to lower the safety factor of the embankment slope before a liquefaction event.

The lower part of the upstream embankment slope was assumed to slide out of the embankment due to foundation liquefaction, removing lateral support for the upstream dam embankment. The safety factor against sliding for the remaining upstream dam embankment slope under saturation and effective stress conditions is 2.46. This result indicates that if liquefaction actually occurred beneath the upstream dam embankment in the assumed location, the dam is not likely to lose its crest elevation. Foundation materials in other elements farther beneath the dam evaluated for liquefaction potential were found to have lower CSR values due to higher effective stresses and lower probability of liquefaction.

Case II

Three scenarios were subjected to slope stability analysis of the upstream dam embankment slope under rapid drawdown conditions. The scenario description and analysis results are described below.

Scenario 1

Scenario 1 is the existing dam and maximum operating reservoir elevations of 1831 feet. Density and shear test strength parameters from 1982 geotechnical investigations described above were used in the Scenario 1 analysis. The safety factor against sliding for the upstream dam embankment slope under rapid drawdown conditions is 1.80.

Scenario 2

Scenario 2 is the proposed dam raise by 5 feet to elevation 1850 feet and proposed maximum reservoir operating elevation raise to 1845 feet (maximum elevation under design inflow flood).

Density and shear test strength parameters from 1982 geotechnical investigations described above were used in the Scenario 1 analysis. The safety factor against sliding for the upstream dam embankment slope under rapid drawdown conditions is 1.70.

Scenario 3

Scenario 3 is the proposed dam and reservoir elevation raises with adjusted soil strength parameters to obtain a safety factor at or very near 1.5. Strength parameters were adjusted to a cohesion value of 800 psf and internal friction value of 30 degrees as describe above under "Adjusted Strength Parameters, Cohesion and Internal Friction." The safety factor against sliding for the upstream dam embankment slope under rapid drawdown conditions is 1.56. WHAT ABOUT THE DOWNSTREAM EMBANKMENT SLOPE?

Outputs for the above Cases I and II, and their scenarios are included in Appendix G.

Pseudostatic Stability Analysis – Determination of Yield Coefficient Ky

Ky Value – Stability Analysis

A pseudostatic stability analysis was performed on the upstream and downstream embankment slopes to determine the yield coefficient Ky. This was done by introducing horizontal ground acceleration values into the analysis to find the horizontal acceleration value that brings the safety factor determined for static conditions to 1.0.

Determination of the Ky value was based on the embankment slope with the lowest safety factor. In this case, the static safety factor of 1.51 for the downstream embankment slope under Case I, Scenario 3 was used for a measure of conservatism. For a safety factor near or equal to 1.0, the related horizontal ground acceleration value is 0.20g. This value is Ky for use in estimating potential amount of embankment deformation due to a seismic event. Results of the stability analysis are presented in Appendix F of this report.

Deformation Potential

A ky value of 0.20 was found to result in a safety factor of 1.00 from pseudostatic stability analysis. The Ky value based on the Bray & Tarasarou approach is 0.042. Since a Ky value of 0.20 is required according to pseudostatic stability analysis to reduce safety factor to 1.00, a Ky value of 0.042 based on Bray & Travasarou will not reduce the safety factor to 1.0. This means the safety factor will be greater than one, meaning in turn that potential dam embankment deformation will be negligible and that performance of the dam under the selected MCE will be satisfactory.

Analysis Step 7 – Cross Check Ky Value with Other Methods

The Ky value determined from the pseudostatic slope stability analysis was compared with Ky values calculated according to the Newmark and Chin-Chin Tsa and Yu-Chien methods. Both of these methods are relatively simplistic.

Newmark Calculation

Newmark's (1965) method of assessing performance of slopes during earthquake shaking is based on a rigid-block analysis. It is used widely in engineering practice to estimate earthquake induced displacement. The Newmark approach includes calculation of an "N" value, corresponding to the Ky value of more recent publications. The equation for calculating the N value is below;

N = (FS-1)(Sine alpha)
Where FS = static factor of safety of the mass and alpha is the angle of the sliding plane with the horizontal.

The FS from stability analysis is 1.51. The angle alpha is 25 degrees and the sine of the angle 25 degrees is 0.423. Accordingly:

N = (1.51-1.0)(0.423) = (0.51)(0.423) = 0.22

Chin-Chin Tsa and Yu-Chun Chien Calculation

Chi-Chin Tsa and Yu-Chun Chien, Assistant Professor, Department of Civil Engineering and Graduate Research Assistant, respectively, both of the National Chung Hsing University, Taichung, Taiwan, developed a simplified procedure that cuts down the iterative, trial and error methods and allows direct evaluation of Ky for both shallow and deep slope failures. The factor of safety of slopes under static conditions is first determined without iterative calculations. The Ky value is then determined through correlations between the static factor of safety and Ky values developed by Tsa and Chien. This method is described in their publication "A Simple Procedure to Directly Estimate Yield Acceleration for Seismic Slope Stability Assessment."

$$Ky = \frac{(FS - 1)}{1/\tan phi + \tan alpha}$$

Where FS = static safety factor of the mass, phi is the internal friction angle of the embankment material and alpha is the angle of the sliding plan with the horizontal.

The FS from stability analysis is 1.51. The adjusted internal friction angle of the embankment material is 30 degrees, the angle alpha is 25 degrees and the tangent of the 25-degree angle is 0.466.

$$Ky = \frac{(1.51-1.0)}{1.733 + 0.466} = 0.51/2.20 = 0.23$$

Ky Comparison Summary

Method	**Ky**
Newmark	0.22
Chi-Chin, Chien	0.23
Spencer Stability Analysis	0.20

Conclusion - Ky Value for Seismic Displacement Analysis

Based on the stability analysis and the comparison of Ky values determined by the Newmark and Chi-Chin Tsa and Yu-Chun Chien, the Ky value of 0.20 is reasonable for the result of the pseudostatic stability analysis conducted for this report.

Analysis Step __ - Cross-Check Seismic Deformation Analysis with Swaisgood

Potential deformation of the dam was cross-checked with the method of deformation analysis reported by J. R. Swaisgood (2003 Pacific Conference on Earthquake Engineering). Figure 15A shows the relationships between peak ground acceleration and crest settlement of a dam as a percentage of the dam height and the thickness of foundation material. The peak ground acceleration at the site is estimated at 0.14g. The peak ground acceleration at the Pelton Reregulating dam was estimated at 0.22g by ___. This dam is approximately 22 km from the Warm Springs Fault, the reported MCE source. Currant Creek dam is located about 64 km from the Warm Springs Fault, a distance that can attenuate peak ground acceleration at that site by a significant amount. Review of Figure 17 (FEMA, Attenuation Relation for Shallow Crustal Earthquakes) shows that attenuation over the 64 km distance from the Warm Springs Fault would result in peak ground acceleration of 0.07g at Currant Creek dam. This attenuation value agrees with relationships between distance from the source earthquake and Magnitude reported by Abrahamson et al. This value is about 32 percent of the peak ground acceleration for the Pelton Reregulating dam.

In an effort of conservatism, the peak ground acceleration at Currant Creek dam was taken for analysis at 0.14g, which is a multiple of 2.0 times the 0.07g value reflected in attenuation data. Based on Swaisgood data as shown on Figure 18A, peak ground acceleration of 0.14g correlates to dam crest settlement of 0.03 percent of the dam height plus foundation material. In this case, the center part of the dam rests on bedrock on the bottom of the core trench. The height of the dam above the bottom of the core trench is 96 feet. Crest settlement at 0.03 percent of the dam height is 0.029 feet, or 0.35 inches, or 0.89 centimeters.

Swaisgood also correlates peak ground acceleration, earthquake Magnitude and estimated dam crest settlement as a percent of the dam height plus the foundation material thickness. This is shown on Figure 18B. For the assumed Magnitude of 7.0 and peak ground acceleration of 0.14g, the estimated settlement is 0.05 percent of the dam height above the bedrock on the bottom of the core trench. For the dam height of 96 feet, the estimated crest settlement is 0.048 feet, or 0.58 inches, or 1.46 centimeters. CHECK THE FIGURE FOR PGA AND MAGNITUDE USED (0.16 G & M=7??)

Swaisgood reports that serious levels of damage were reported only in instances where the peak ground acceleration exceeded 0.2g. Investigation by Seed, Makdisi and DeAlba, 1978, found evidence "that well-built dams can withstand moderate shaking with peak accelerations up to at least 0.2g with no harmful effects."

Conclusion

Results of all methods of deformation analysis show that potential deformations in the dam are negligible. The results indicate that the Currant Creek dam can withstand moderate shaking with no harmful effects.

Analysis Step 6 - Liquefaction Analysis

Approach

Liquefaction potential was evaluated according to guidelines presented in the U.S. Bureau of Reclamation "Design Standards No. 13, Embankment Dams, Chapter 13: Seismic Analysis and Design Phase 4: Final, May 2015 (herein after referred to as Chapter 13). Additional supplemental information was used from various cited sources referenced in the following analysis description.

Liquefaction potential was evaluated for the foundation materials of the existing dam. The dam embankment is not susceptible to liquefaction due to the densely compacted, cohesive and granular nature of the embankment fill.

Key evaluation steps include the following:

- Determination of engineering properties of soils based on evaluation of laboratory tests completed on representative soil samples during the 1982 design work for the existing dam. Evaluation of test results focused on grain size distribution including percent passing the No. 4, No. 40 and No. 200 U.S. Standard Sieve sizes, Atterberg Limits for liquid limit, plastic limit and plasticity index, in-place dry density and moisture content, maximum dry density and optimum moisture content, and degree of compaction of soils at their natural dry density and moisture content (estimated in some cases).

- Evaluation of seismic velocity data obtained with seismic refraction surveys during the 1982 geotechnical investigations.

- Geologic characterization of the Currant Creek floodplain materials and related materials in the dam foundation. This included preparation of a cross-section of stratigraphic conditions in the floodplain prior to dam construction and stratigraphic conditions after excavation of the dam foundation and stripping of organic and unsuitable surface soils beneath the dam prior to fill placement thereon.

- Identification of liquefaction evaluation parameters, including:

    - Peak seismic ground acceleration at the dam site (USGS, ASCE7-16), maximum considered earthquake.
    - Total overburden stress on elements located in dam foundation materials.
    - Total effective overburden stress on elements located in dam foundation materials.

- Maximum induced seismic shear with depth.
- Cyclic Stress Ratio CSR (measure of seismic force contributing to liquefaction).
- Cyclic Resistance Ratio CRR (measure of material resistance to liquefaction).
- Plasticity Index of soils.
- Percent of soil samples passing the No. 4, No. 40 and No. 200 U.S. Standard Sieves.
- Shear wave velocity values for foundation materials using seismic refraction survey data collected during the 1982 engineering work.
- Correlations between CSR, shear wave velocities and liquefaction potential.

- Application of the parameters in calculation and analysis of liquefaction potential for elements located in dam foundation materials.

Engineering Properties of Soils Based on Laboratory Tests

Engineering properties of soils in the dam foundation area were determined first in order that these properties are included in the geologic stratigraphic analysis for the Currant Creek floodplain in the area of the existing dam. Engineering properties relevant to liquefaction potential were determined by review and analysis of results of laboratory tests performed on representative samples during the 1982 engineering work for dam design. Engineering properties compiled through this review are shown on Table 1 (Appendix G).

Soil properties relevant to liquefaction potential include: 1) percent passing the No. 4 U.S. Standard Sieve; 2) percent passing No. 40 U.S. Standard Sieve; 3)percent passing the No. 200 U.S. Standard Sieve; 3) Plasticity Index; 4) in-place density and moisture content of soils; 5) degree of compaction of the in-place, undisturbed soils; and 6) shear wave velocity.

Liquefaction Susceptibility of Soils

Based on the above-referenced Chapter 13, liquefaction susceptibility applies primarily to noncohesive (granular) soils, such as clean sands or sand-gravel mixtures, silty sands with nonplastic fines, nonplastic silts, etc. Clayey soils are generally more resistant to liquefaction. Accordingly, the first step in evaluation of liquefaction potential was to consider liquefaction potential of the prevailing dam foundation soils relative to their clay-like or granular characteristics.

Fine-Grained and Clay-like Soils

Liquefaction can occur in clayey soils and where it has occurred, it has been near the ground surface, where effective preconsolidation stress is low and the soils have not been consolidated to a low void ratio and low water content. However, beneath a dam embankment, clayey soils are generally consolidated to water contents well below their liquid limit, and much less susceptible to liquefaction (Chapter 13). If they do reach a state of liquefaction, most likely the state is in condition of cyclic mobility, rather than flow liquefaction. Cyclic mobility is described as "a category of soil liquefaction in which strains tend to be limited by increases in shearing resistance following an initial nearly complete loss of effective stress and shearing resistance. This occurs in medium-density granular soils, and it may help to explain why liquefaction has occurred without instability or very large deformations" (U.S. Bureau of Reclamation, Chapter 13: Seismic Analysis and Design, Page 13-3.

Boulanger and Idriss (2004) distinguish clay-like soils from granular soils according to the Plasticity Index (PI) of the clayey material. Their distinction on this basis is that a clay-like soil would have a PI of 4 to 7. A higher PI indicates more claylike behavior. Chapter 13 also suggests that clayey sands and clayey gravels could also behave as fine-grained clay-like soils if more than 50 to 60 percent passes the No. 40 U.S. Standard Sieve. It is noted that these distinctions between clay-like and granular soils are for choosing an appropriate engineering procedure for evaluating stress-strain behavior, not a criterion for ruling out liquefaction potential.

Research of a large set of case histories and laboratory testing by Bray and Sancio (2006) led to their following conclusions in regard to liquefaction potential of fine-grained and clayey soils relative to PI and Liquid Limit:

1. If the PI is greater than 18, the soil may be considered non-liquefiable.
2. If the PI is greater than 12, the soil may be considered non-liquefiable, provided that the water content is less than 85 percent of the Liquid Limit (LL).
3. If a soil does not have some plasticity, even if the PI is less than 12, it may be considered non-liquefiable, provided that the water content is less than 80 percent of the LL.

Table 1 (Appendix G) shows soils classified as clay-like based on laboratory testing results. The table also shows that nearly all of the clay-like soils are not susceptible to liquefaction. This finding is based on the PI and the natural water content of the samples in comparison to the Liquid Limit for the samples. The relationship between natural water content of samples and their Liquid Limit is summarized below:

SAMPLE	NAT. WATER CONTENT (%)	LIQUID LIMIT (%)	PI	NAT. WATER CONTENT AS PERCENT OF LIQUID LIMIT
T8-1	16.3	43	10	38
T9-1 (Granular)	14.8	50	19	30
T107-2	23	59	29	39
T109-5	39.9	49	27	81
T110-2	33.5	44	21	76

Application of the three Bray and Sancio (2006) criteria to the above natural moisture content and plasticity indices indicate that these samples can be considered as non-liquefiable.

Granular Soils

Granular soils are defined as soils lacking in appreciable plasticity and that behave in a sand-like manner, rather than clay-like manner under seismic loading. Boulanger and Idriss (2004) consider that a PI less than 4 to 7 would reflect a granular soil. Some soils classified as silt (ML) and high-plasticity silt (MH) may need to be treated as granular soils.

Three of the tested soil samples shown on Table 1 are classified as granular soils. This is based primarily on grain-size distribution and percent passing the No. 4, No. 40 and No. 200 U.S. Standard Sieves.

The sample B-4 test results (Table 1) show that 48 percent of the sample by weight consists of gravel to cobble-size material, with 52 percent passing the No. 4 sieve, 31 percent passing the No. 40 sieve and 21 percent passing the No. 200 sieve. The PI is 7, consistent with a granular soil classification. The soil is classified as silty sand (SM) according to the Unified Soil Classification System (USCS).

The sample T9-1 test results (Table 1) show that 32 percent consists of gravel to cobble-size material, with 67 percent passing the No. 4 sieve, 37 percent passing the No. 40 sieve and 25 percent passing the No. 200 sieve. The PI is 19, indicating plastic fines impart a clay-like quality

The sample T13-3 test results (Table 1) are only for natural density and moisture content. The granular classification is based on descriptions on the exploratory trench log prepared by an engineering geologist during the 1982 geotechnical investigations. The log describes the material as clayey gravel, wet, dense, coarse gravel to cobbles.

Soil Density Evaluations

Table 2 (Appendix G) was prepared in evaluations of soil density conditions and susceptibility to liquefaction. The table summarizes laboratory test results relative to natural dry density, maximum dry density, specific gravity, void ratio, percent passing the No. 4 sieve and percent passing the No. 200 sieve. The table also shows the USCS classification, liquefaction analysis classification (clay-like, granular) and estimated relative density values correlated with Standard Penetration Resistance (STP) values.

Natural dry density and moisture content were determined by laboratory tests for 9 of the 13 samples shown on Table 2 (columns 7 and 8). Maximum dry density and optimum moisture content tests were performed on 3 samples (column 9 and 10). Specific gravity determinations were made on 6 of the 13 samples shown on Table 2 (column11). Specific gravity was estimated for 2 samples as shown on the table.

Using the natural dry density and specific gravity for the tested samples, the void ratio at the natural density of the samples was determined. When a soil is compacted to its maximum dry density, the void ratio will decrease. The amount of potential void ratio decrease due to compaction was determined those samples tested for natural dry density, maximum dry density and void ratio. The void ratio calculated for the maximum dry density condition was subtracted from the natural density condition to obtain the amount of void ratio deduction.

Natural density void ratios were calculated for those samples tested for their natural dry density and specific gravity. The natural density void ratios are shown in column 12. Maximum dry density void ratios were calculated for those samples tested for maximum dry density, and specific gravity. The maximum dry density void ratios are shown in column 13 for sample T9-1 and T110-2.

The reduction in void ratio from soil compaction to its maximum dry density is shown in column 14 for sample T9-1 and T110-2. The void ratio reduction for sample T9-1 is 30 percent. The void ratio deduction for sample T110-2 is 44 percent. Sample T9-1 is a granular soil and sample T110-2 is a clay-like soil. The lessor void ratio reduction for sample T9-1 is consistent with expectations for a granular soil. The higher void ratio reduction for sample T110-2 is consistent with expectations for a clay-like soil. These respective void ratio reduction factors were used to estimate maximum dry density for other samples for which natural dry density tests were done without tests for maximum dry density. Specific gravity values were estimated for those samples not tested for specific gravity.

Maximum dry density void ratios were estimated by application of the void ratio reduction factor for granular soils or for clay-like soils to the natural dry density void ratio. For granular soils, the natural dry density void ratio was reduced by 30 percent. For clay-like soils, the natural dry density void ratio was reduced by 44 percent. The maximum dry density for the samples was then estimated according to the reduced void ratio and specific gravity. The estimated maximum dry density values for various samples are shown in column 16 and estimated degree of soil compaction in its natural condition is shown in column 17.

Relative density can be correlated with Standard Penetration Resistance (STP) blow counts, which can provide an indication of liquefaction potential. The relative density (Dr) is a function of the natural dry density, maximum dry density and minimum dry density of a soil. Natural dry density and maximum dry density values are available based on laboratory tests as described above. For other samples, maximum dry density was estimated. For all samples, the minimum dry density was estimated. The results are shown in column 18 of Table 2.

Cyclic Stress Ratio

The cyclic stress ration (CSR) is a measure of loading applied to soil by an earthquake. The CSR is determined by the equation:

> CSR = 0.65 X peak cyclic shear stress caused by earthquake divided by the vertical effective stress (pre-earthquake).

The peak cyclic shear stress = peak horizontal ground acceleration at the ground surface X total overburden stress X a stress reduction coefficient as a function of depth below the ground surface.

The peak horizontal ground acceleration at the site is 0.16g for the Maximum Considered Earthquake (MCE). This is based on the USGS-ASCE7-16 web-based tool and the latitude and longitude of the dam site.

The CSR was calculated for elements located at four locations in the foundation soils of the existing dam as shown on the cross-section of Figure 19. CSR values for each element were calculated for existing reservoir conditions and for proposed reservoir conditions. The existing reservoir condition is a maximum water surface elevation of 1,831 feet. The proposed condition is a maximum water surface elevation of 1,845 feet with a dam raise of 5 feet for freeboard. Calculated CSR values are below:

ELEMENT	CONDITION	LOCATION	CSR
1A	Existing Reservoir	Below mid-upstream dam slope	0.21
1B	Proposed Reservoir	Below mid-upstream dam slope	0.24
2A	Existing Reservoir	Below upper downstream dam slope	0.11
2B	Proposed Reservoir	Below upper-downstream dam slope	0.14
3A	Existing Reservoir	Below mid-downstream dam slope	0.10
3B	Proposed Reservoir	Below mid-downstream dam slope	0.14
4A	Existing Reservoir	Below upstream dam slope near toe	0.31
4B	Proposed Reservoir	Below upstream dam slope near toe	0.49

The higher CSR values for Element 4A and 4B are due to increasing reservoir water depth over the upstream dam embankment slope and corresponding reduction in effective overburden stresses.

Application of Cyclic Stress Ratio in Evaluating Liquefaction Potential

The Cyclic Stress Ratio (CSR) is used in conjunction with other parameters to evaluate liquefaction potential of soils. CSR has been correlated with Standard Penetration Test (STP) data and with shear velocity (Vs) data for soil materials. Shear wave velocity has been correlated with bulk soil density.

Standard Penetration Tests were not performed during the 1982 geotechnical investigations. Undisturbed chunk samples were tested for natural dry density and moisture content. Seismic refraction surveys were conducted in the foundation area of the dam and spillway areas. The locations of three seismic profiles B-B', F-F' and E-E' in the dam area are shown on the cross-section of Figure 17 (Geology of Alluvial Deposits in Dam Foundation). These surveys indicate seismic velocities in the alluvial materials range between 1,400 to 6,300 feet per second. These velocities are for compressional, or P-waves. Shear wave velocities are less than P-wave velocities. For analysis of liquefaction potential using the CSR, it is necessary to determine representative shear wave velocities.

Bulk Density Values

Bulk density of soils has been correlated with shear wave velocity for soils (P. Arbazhagan, et al, Journal of Geophysics Engineering 13) and provides useful wave velocity information for evaluating liquefaction potential of the dam foundation soils.

Natural dry density and moisture content values for tested samples were converted to bulk density values for correlation with shear wave velocities. These conversions are shown below.

SAMPLE	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	BULK DENSITY (gm/cc)	SOIL TYPE
B-3A	73.5	49	1.75	CLAY-LIKE
T8-1	80.2	16.3	1.49	CLAY-LIKE
T9-1	104.1	14.8	1.91	GRANULAR
T9-3	89.4	24.4	1.78	CLAY-LIKE
T13-2	73.5	21	1.42	CLAY-LIKE
T13-3	90.2	23	1.78	GRANULAR
T107-2	97.9	23	1.93	CLAY-LIKE
T109-5	80.2	39.9	1.80	CLAY-LIKE
T110-2	79.9	33.5	1.71	CLAY-LIKE

The maximum and minimum bulk densities are 1.93 and 1.42, respectively. The average bulk density for all soil samples is 1.73. The bulk densities for the granular soil samples are 1.78 and 1.91. The bulk densities are in units of gram/cc for correlation with shear wave velocity based on the Arbazhagan, et al work.

The samples of the granular soil type were obtained from exploratory pits T9-1 and T13-3, both located about 1,500 feet upstream of the dam, in alluvial deposits of Currant Creek. The exploratory pit logs describe the observed and sampled soils as medium dense clayey gravels (GC) to sandy gravels. The sample T9-1 was later classified as clayey sand (SC) based on gradation testing. The Atterberg Limits for sample T9-1 are liquid limit of 50, plastic limit of 21 and plasticity index of 19.

Although the two granular soil samples are from pits a significant distance upstream of the dam, they are similar to descriptions of materials observed in exploratory pits and borings in the alluvial materials at the dam site. Exploratory pits T-109 and T-110, and borings B-201, 202, 203 and 204 were excavated in alluvial materials inside the dam footprint.

The log for pit T-109 (Figure 20) describes the observed soils as medium dense gravel to sand in the upper 5.5 feet, overlaying clay with gravel and sand lenses, overlaying blue clay at the 15-foot pit bottom. All of this material was removed to bedrock in the foundation core trench excavation.

The log for pit T-110 (Figure 20) describes observed soils as medium dense to dense fine gravels to small cobbles and medium to coarse sand overlaying very silty material with sand and gravel trace to an 11-foot depth. All of this material was removed to bedrock in the foundation core trench excavation.
The geologic log for Boring 201 shows soft to firm, silt to clay mixtures with sand and gravel trace to depth of 16 feet, with a 2-foot thick layer of gravel to cobble-size rock, at 7 to 9 feet, overlaying gravelly to very gravelly sand to bedrock at 25-foot depth. The upper approximate 5 feet of material was removed during excavation of the foundation core trench, leaving gravel to cobble-size rock in a silt to clay matrix, overlaying silt to clay mixtures with sand and gravel trace, overlaying very gravelly sand to bedrock at the 25-foot depth.

The geologic log for Boring 202 (Figure 20) shows medium dense gravel to sand mixture with minor silt and clay lenses, cobbles intermixed at 4.5 to 7-foot depth, softer at 7-foot depth, gravelly below 10 feet to bedrock at 15-foot depth. The upper approximate 8 feet of material was removed during excavation of the foundation core trench, leaving medium dense, gravel to sand mixture with minor silt and clay lenses, gravelly at 10-foot depth, extending 7 feet to bedrock.

The geologic log for Boring 203 shows medium dense, well-graded gravel-sand-silt mixture to 8-foot depth, overlaying clayey, gravelly mixture to bedrock at depth of 18 feet. The upper approximate 8 feet of material was removed during excavation of the foundation core trench, leaving the clayey, gravelly mixture in the approximate 10-foot depth between the core trench excavation bottom to bedrock.

The geologic log for Boring 204 shows medium dense, gravel-sand-silt mixture (very gravelly) with clay and sand matrix at 13-foot depth, extending to bedrock at the 19-foot depth. The upper approximate 8 feet of material was removed during excavation of the foundation core trench, leaving about 5 feet of medium dense gravel-sand-silt mixture overlaying about 6 feet of gravelly material with clay and sand matrix, and boulders, over bedrock at a depth of 19 feet.

Common to all the above pit log descriptions, including those for T9-1 and T13-3, is the gravel fraction of the soil mix, with a matrix of clay, silt and sand, in a medium dense condition. The nature of these materials is illustrated on the cross-section of Figure 17. All geologic logs for exploratory pits excavated in the alluvial deposits of Currant Creek show prevalence of medium dense, gravelly, sandy, silty materials with variable amounts of clay from none to enough to reflect clayey gravel, gravelly clay, to clayey sand descriptions. In some cases, relative thin interbeds of sands, silts and “clay pods” are reported. In some cases, stratigraphic layers are discontinuous, consistent with repetitive stream deposition and erosion cycles. However, the gravel fraction is persistent in the alluvial deposits (Figure 20).

Considering the above descriptions of the materials encountered in exploratory pits and borings and the bulk density values for two samples of gravelly, clayey, sandy materials, it is reasonable to expect that the bulk density values reflect general density conditions for the granular materials in the dam foundation area. The bulk density values from the laboratory tests are 1.78 and 1.91 grams/cubic centimeter. The average of these two values is 1.85 grams/cubic centimeter. The average for all tested samples is 1.73 grams per cubic centimeter. For purposes of this report, the bulk density is taken at 1.73 gm/cubic centimeter.

Correlations – Bulk Density with Shear Wave Velocity

Shear wave velocities for use with CSR in evaluating liquefaction potential were estimated by correlations between soil bulk density and shear wave velocities according to work by P. Arbazhagan, et al, as cited above. Graphs correlating soil bulk density with shear wave velocity based on Arbazhagan, et al, work for all soil types, fine-grained soils and for coarse-grained soils are shown on Figure 21.

In one case for this report, the average bulk density of 1.73 gm/cc for all tested samples was used in the correlation. This overall average was used in correlation with shear wave velocity and bulk density of coarse-grained soils, finding that the shear wave velocity is about 275 m/sec. Shear wave velocities for the actual bulk densities of sample T9-1 and T13-3 are about 380 m/sec and 285 m/sec, respectively. All of these correlations are shown on Figure 21 (graph from Arbazhagan).

No natural dry density/moisture content test was conducted on sample T6-1 from an exploratory pit about 1,900 feet upstream of the dam such that no bulk density value is available No Plasticity Index is also available. The gradation test indicates the soil is silt (ML) with 93 percent passing the No. 4 sieve and 52 percent passing the No. 200 sieve. The soil is classed as a clay-like soil. Without bulk density and Plasticity Index, evaluation of liquefaction potential of this soil is difficult. Silt (ML) soils encountered in borings and exploratory pits at the dam site were in shallow deposits in the upper 4 feet of depth below the ground surface. These silts were removed during foundation excavation and ground preparations in the dam site.

Silt layers reported on logs of exploratory pits excavated in the Currant Creek alluvial deposits were encountered generally in the upper 2 to 4 feet below the ground surface, overlying intermixed gravelly, sandy, silty and clayey materials. In one case, silt was encountered to a depth of about 6 feet below the ground surface. In some cases, clays are also reported in the upper approximate 1 to 3 feet below the ground surface, as layers or as a fraction of the silt soils. In one case, clay soils are described to a depth of about 9 feet. In another case, a 3-foot-thick layer of clay (CL) is reported at a depth of 6 to 9 feet.

Estimated Liquefaction Potential

The liquefaction potential of the dam foundation materials was estimated according to the nature of the materials beneath the dam as described above under ***Bulk Density Values*** and ***Correlations – Bulk Density with Shear Wave Velocity*** and by correlation of the shear wave velocity with CSR.

Correlations between CSR and shear wave velocity are shown graphically on Figure 22, showing liquefaction probability as a function of shear wave velocity. This source for this figure is "Design Standards No. 13, Embankment Dams, Chapter 13: Seismic Analysis and Design, Phase 4: Final", U.S. Department of the Interior, Bureau of Reclamation, Appendix E, Figure E2. This graph is based on an earthquake Moment Magnitude of 7.5 and effective overburden pressure of 100kPa (2,089 psf). The Moment Magnitude of 7.5 is taken as reasonable for liquefaction analysis considering determination according to USGS ASCE7-16 tool of peak ground acceleration and other seismic parameters for the dam site based on the Maximum Considered Earthquake. Overburden pressures for the 4 Elements analyzed for CSR ranged from a low of 1,404 to 6,981 psf. The low value is for the Element 4, near the toe of the upstream slope of the dam embankment. The minimum effective stress value for the remaining Elements 1, 2 and 3 is 2,216 psf, such that they are all higher than the 2,089 psf value on which the Figure 19 correlations are based. The higher effective stress values contribute to lower calculated CSR values, meaning lower measure of seismic loading imposed on the soil. Accordingly, Figure 19 is taken as reasonable for this analysis.

For an average shear wave velocity of 275 m/sec for all soils (Figure 19) and the maximum CSR value of 0.49, the intersection of these two points is in the "no liquefaction" area of the Figure. The intersections for the remaining, lower CSR values in the range of 0.10 to 0.31 and shear wave velocity are also in the "no liquefaction" area of the Figure.

The bulk density for sample T13-2, from an exploratory pit about 1,500 feet upstream of the dam, is 1.42 gm/cc. This is the lowest bulk density for the tested samples. The soil is classified as silt (ML) based on the exploratory pit log. Correlation of the bulk density with shear wave velocity indicates the shear wave velocity would range from roughly 100 to 150 m/sec. The intersection of CSR and shear wave velocity on Figure 22 is within the "liquefaction" area of the figure. The silt from which this sample was taken is in a layer within the upper 3 feet below the ground surface. As described for the silt in sample T6-1, the only silts encountered in borings and exploratory pits in the dam site area were also shallow and removed during foundation excavation and ground preparations.

Assumed Liquefaction at Element 1 Near Toe of Upstream Dam Embankment

This slope stability analysis was done on the assumption that a lower part of the downstream dam embankment slope was displaced by liquefaction of foundation materials. Part of the embankment slope was removed, followed by slope stability analysis with STABL 6. The purpose was to estimate the effect of partial embankment slope removal on overall stability of the upstream dam embankment slope.

The soil strength parameters of the upstream dam embankment slope were 800 psf for cohesion and 30 degrees for internal friction angle. These values were lowered for a conservatism measure from the laboratory triaxial shear test values of 1,800 psf and 12 degrees for cohesion and internal friction angle, respectively. The results show that the safety factor for the remaining upstream dam embankment slope is 2.46, indicating low potential for the dam to lose its crest. Outputs for the STABL 6 analysis are in Appendix F.

GEOTECHNICAL IMPLICATIONS – STORAGE INCREASE BY RAISING EXISTING DAM

Increase in Reservoir Storage Capacity

Capacity to store an additional estimated 450 acre-feet of water in the Currant Creek Reservoir will require raising the crest of the spillway control section from its current elevation of 1,831 feet to an elevation of 1,841 feet. A labyrinth weir is planned for raising the spillway control section. The depth of flow over the weir during the PMF inflow design storm is 4 feet. The maximum reservoir water surface elevation under this flow condition is 1,845 feet, the current top elevation of the dam. Five feet of freeboard at the dam will be provided by raising the dam to a top elevation of 1,850 feet. The actual top elevation near the center of the dam is planned at 1850.5, tapering to 1,850 feet at each end, allowing for camber. A cross-section of the existing and proposed dam configuration at the maximum section, including proposed toe drain modification, are shown on Figure 8.

Raising of the maximum normal reservoir operating elevation to elevation of 1,845 feet will require raising of a segment of the Muddy Road. This is a Wasco County road and the proposed raise will be completed in accordance with applicable county requirements. A number of existing culverts will also require raising. The road segment and culverts to be raised and the amount of raising are shown on Figure 23.

Geometry of Additional Dam Embankment

The minimum width for the top of the embankment fill to be added to the existing dam will be no less than 15 feet. This top width can be achieved by extending the upstream slope upward at the existing 3 to 1 angle and extending the downstream slope upward at the existing 2 to 1 angle as shown on Figure 8.

Loading of Existing Dam by Added Embankment Fill

Materials proposed for raising the existing dam consist generally gravel-silt-clay-sand mixtures. Much of these materials are anticipated to come from excavation of colluvial and alluvial materials from the new emergency spillway channel. Maximum dry density and optimum moisture content values are anticipated to range from 110 to 120 pcf and 12 to 15 percent, respectively, considering results of tests conducted during the 1982 geotechnical investigations. For this analysis, a maximum dry density of 115 pcf at an optimum moisture content of 14 percent were assumed for the material to raise the dam. Assuming compaction of the topping fill at 90 percent of the maximum dry density according to Modified Proctor method, at the optimum moisture content, the wet density of the compacted material would be 118 pcf.

Considering the depth of new embankment fill at 5 feet and maximum anticipated fill density of 118 pcf , the additional fill load applied to the existing dam would be about 590 psf.

Settlement Potential under Proposed Embankment Load

Considering the generally well-graded mix of gravels-silts-clays-sands in the make-up of the fill materials in the existing dam embankment, the time span of 36 years since its construction in 1983, relatively low post-construction settlement potential and relatively low settlement potential at the current time, it is unlikely that additional settlements that are significant to the dam will occur under the proposed added fill load. The height of the existing dam at the maximum section is 80 feet.

Settlement of the existing dam is negligible based on comparison of recent topographic surveys and related elevations determined for the top of the existing dam with original design elevation data. Accordingly, it is expected that settlements under the proposed loading of 5 feet of embankment fill will result in settlements that are insignificant to the dam.

Considering the High Hazard classification of the dam, camber provisions are reasonable in raising the existing dam. This can be done by raising the center of the existing dam by 5.5 feet, sloping to 5.0 feet at each end of the dam.

Influence of Dam Raise on Slope Stability

The safety factors calculated in 1982 for the upstream and downstream slopes of the existing dam embankment are both equal to 1.6. The safety factors were calculated by hand according to a “modified Swedish slip circle” method during design of the existing dam.

Stability analysis was updated for purposes of the current project by use of STABL 6 software as described on pages 38 to 40 of this report. Results of slope stability analyses for Cases I and II and the various scenarios for each Case are described on pages 40 to 42 of this report.

In summary, the safety factor for the upstream dam embankment under existing conditions and rapid drawdown is 1.81. The safety factor for the downstream embankment under existing conditions and steady state seepage is 1.83.

Stability analysis was also performed for the proposed raised dam condition. The safety factor for the upstream dam embankment for the proposed condition, under rapid drawdown, is 1.70. The safety factor for the downstream embankment under proposed dam raise and raise of maximum reservoir operating elevation, and steady state seepage is 1.56 (Case I, Scenario 2).

Minimum safety factors recommended (USBR, Embankment Dams) for steady state seepage conditions and for rapid drawdown conditions are 1.3 and 1.1 to 1.3, respectively. For this proposed project, a minimum safety factor of 1.5 is desired for the downstream embankment slope.

Addition of 5 feet of compacted fill to the crest of the dam and raising the maximum reservoir operating elevation by 14 feet did not reduce safety factors below 1.5 based on STABL 6 analysis.

Seepage through the Dam and Foundation

Seepage through the dam is minimal based on observed spring-time discharge from a toe drain pipe near the south abutment. The lack of saturated ground near the toe of the dam and in areas downstream of the dam suggest that foundation leakage is also minimal.

Permeability of the representative soil for the dam embankment was estimated to be 0.0005 feet per day (Freeze and Cherry, 1979) during original design work in 1982.

Estimated Seepage for Existing Conditions (Permeability Estimated at 0.0005 ft/day)

Seepage through the dam for existing maximum reservoir water elevation of 1,831.0 feet was estimated with a flow net constructed according to criteria presented by Harry Cedergren in his book "Seepage, Drainage, and Flow Nets," 1977.

Seepage through the dam can be estimated from a flow net by the equation Q = (kh)(nf/nd):

Where
Q = total seepage through a flow net, per lineal foot of seepage face
k = permeability in feet per day
h = net head in feet (head between reservoir level and foundation)
nf = number of flow channels in flow net
nd = number of head drops in flow net

The estimated total discharge through the toe drain is about 29 gallons per day based on the k value of 0.0005 feet per day assumed in the 1982 geotechnical investigations, h value of 77 feet, nf/nd ratio of 0.48 based on the flow net shown on Figure 24, and total estimated 210-foot length of the existing toe drain along the toe of the dam. This amounts to an average rate of about 0.02 gallons per minute.

The flow net was also used to estimate the hydraulic gradient for seepage through the dam. The maximum hydraulic gradient is estimated at 0.25 for the existing maximum normal reservoir water elevation of 1,831.0 feet.

Estimated Seepage for Proposed Conditions (Permeability Estimated at 0.0005 ft/day)

Increasing storage capacity in the reservoir will raise the normal maximum operating elevation of the reservoir surface by 11.5 feet, to an elevation of 1,841.5 feet. The estimated total discharge through the toe drain for the proposed reservoir condition is about 35 gallons per day based on the k value of 0.0005 feet per day, an h value of 88 feet, nf/nd ratio of 0.51 based on the flow net shown on Figure 22, and total estimated 210-foot length of the existing toe drain along the toe of the dam. This amounts to an average rate of a little over 0.02 gallons per minute.

The flow net on Figure 22 for proposed reservoir head conditions was used to estimate the hydraulic gradient for seepage through the dam. The maximum hydraulic gradient is estimated at 0.37 for the proposed maximum normal reservoir water elevation of 1,841.5 feet.

Estimated Change in Seepage and Hydraulic Gradient from Existing to Proposed Reservoir Head Conditions

Total seepage discharge increases by about 21 percent for the proposed reservoir head condition. The hydraulic gradient increases by about 48 percent for the proposed reservoir head condition.

Permeability Adjustments

The above estimated discharge rates are relatively low; however, they are generally consistent with observed conditions on the downstream slope of the dam, the toe area of the dam, and ground surface conditions in alluvial materials downstream of the dam. Estimated seepage rates were cross-checked for prudence, by use of published typical permeability values for clayey, sandy gravels and for clayey sands, which are generally representative of dam embankment materials.

Permeability values for clayey sandy gravels ranges from about 0.014 to 0.0014 feet per day (reference). The average of this range is 0.0077 feet per day. Permeability values for clayey sands ranges from about 0.0014 to 1.57 feet per day. The average of this range is 0.79 feet per day.

Estimated Seepage for Existing Conditions – Adjusted Permeability

For existing conditions, a k value of 0.0077 feet per day is taken as the average value for clayey, sandy gravels. For an h value is 77 feet, nf/nd ratio of 0.48 based on the flow net analysis on Figure 21, and total estimated length of 210 feet for the existing toe drain, the estimated total discharge through the toe drain is about 447 gallons per day. This amounts to an average rate of about 0.31 gallons per minute.

The k value was then adjusted to an average value of 0.79 feet per day for clayey sands, again for existing conditions. The total estimated discharge through the 210-foot length of the existing toe drain is about 45,865 gallons per day. This corresponds to an average rate of about 32 gallons per minute

Estimated Seepage for Proposed Conditions – Adjusted Permeability

For proposed conditions, a k value of 0.0077 feet per day is taken as the average value for clayey, sandy gravels. For an h value of 88 feet, nf/nd ratio of 0.51 based on the flow net analysis on Figure 25 and total estimated length of 210 feet for the existing toe drain, the estimated total discharge through the toe drain is about 543 gallons per day. This amounts to an average rate of 0.38 gallons per minute.

The k value was then adjusted to an average value of 0.79 feet per day for clayey sands, again for proposed conditions. The total estimated discharge through the 210-foot length of the existing toe drain is about 55,693 gallons per day. This corresponds to an average rate of about 39 gallons per minute

Conclusions – Estimated Seepage Rates

Considering that very little discharge has been observed from the southern-most toe drain discharge pipe during short, spring-season periods, and the above estimated seepage discharge rates, the permeability of 0.0005 feet per day estimated in 1982 is reasonably consistent with existing observed conditions. The adjusted average permeability of 0.0077 feet per day for clayey, sandy gravels based on published information, also is reasonably consistent with existing observed conditions. The adjusted average permeability of 0.79 feet per day for clayey sands based on published information, leads to a significantly high seepage rate and is not consistent with existing observed conditions.

Although it is also reasonable to conclude that the existing toe drain and pipe outlet system is adequate to discharge potentially increased seepage rates resulting from the proposed raising of the maximum operating elevation of the reservoir, it is recognized that predictability of seepage under a significantly increased hydraulic gradient is with uncertainties as reflected in the above estimates of seepage. It is concluded that additional drainage provisions are warranted for the dam.

Drainage Provisions

Toe Berm

Excavation of the revised spillway channel will generate earth material that will need to be disposed of at a suitable site. The toe area of the existing dam is in close proximity to the excavation areas and provides opportunity for disposal of excess materials with minimal haul distance. Placement of excess material against the toe and lower part of the downstream dam embankment provides a beneficial support buttress for the downstream dam embankment and minimizes haul costs to a disposal site. Accordingly, provisions are in order to place the excess material in a toe berm against the dam.

Toe berm Drain System

Considering potential for increased seepage through the dam and through the foundation downstream of the core trench, provisions to carry discharge water beneath the toe berm are warranted for the project. In this regard, a trench drain composed of sand meeting the Oregon Department of Transportation (ODOT) gradation specifications for Portland cement fine concrete aggregate according to Table 02690-5 of the 2018 Standard Specifications should be placed against the downstream end of the existing toe drain, extending the full length of the downstream toe of the dam from abutment to abutment.

Sand meeting the above-referenced gradation is a tentative consideration for both filtration and permeability needs, however, the sand gradation for these needs must be verified when fill materials are being screened for use as toe berm embankment fill. The gradation must also be verified for the natural ground soils into which the trench drain system will be installed.

The drain trench should be at least 3 feet wide and excavated to a minimum depth of 5 feet below the existing ground surface along the toe of the existing downstream dam embankment. Excavation of the trench should include excavation to expose the existing toe drain in the dam embankment and the three existing toe drain discharge pipes.

The existing three toe drain discharge pipes should be extended to free, unobstructed discharge points outside the toe of the berm. Extension of the pipes shall be by placement in individual trenches that extend from the toe of the dam to daylighting discharge points outside the toe berm embankment.

The discharge ends of the drain pipes should be unobstructed and readily visible for monitoring purposes. The trenches for the pipes should be at least 3 feet deep below the adjacent ground surface and filled with wet, compacted sand as described above to a depth of 1 foot above the trench bottom. The drain pipes should be placed on top of this sand fill and the remaining trench should be backfilled to the top with wet, compacted sand. Compaction of all sand shall be with a vibratory plate compactor.

The trench excavated along the toe of the dam should be backfilled with wet, compacted sand conforming to the ODOT gradation requirements for Portland cement fine aggregate for concrete. Compaction of the sand should be by vibratory plate in uncompacted lifts of 1 foot or less. The sand backfill should be vertically extended to a position against the existing toe drain material such that hydraulic continuity is established between the existing toe drain and the recommended trench drain beneath the toe berm.

BORROW AREA LOCATIONS

Borrow Material Need

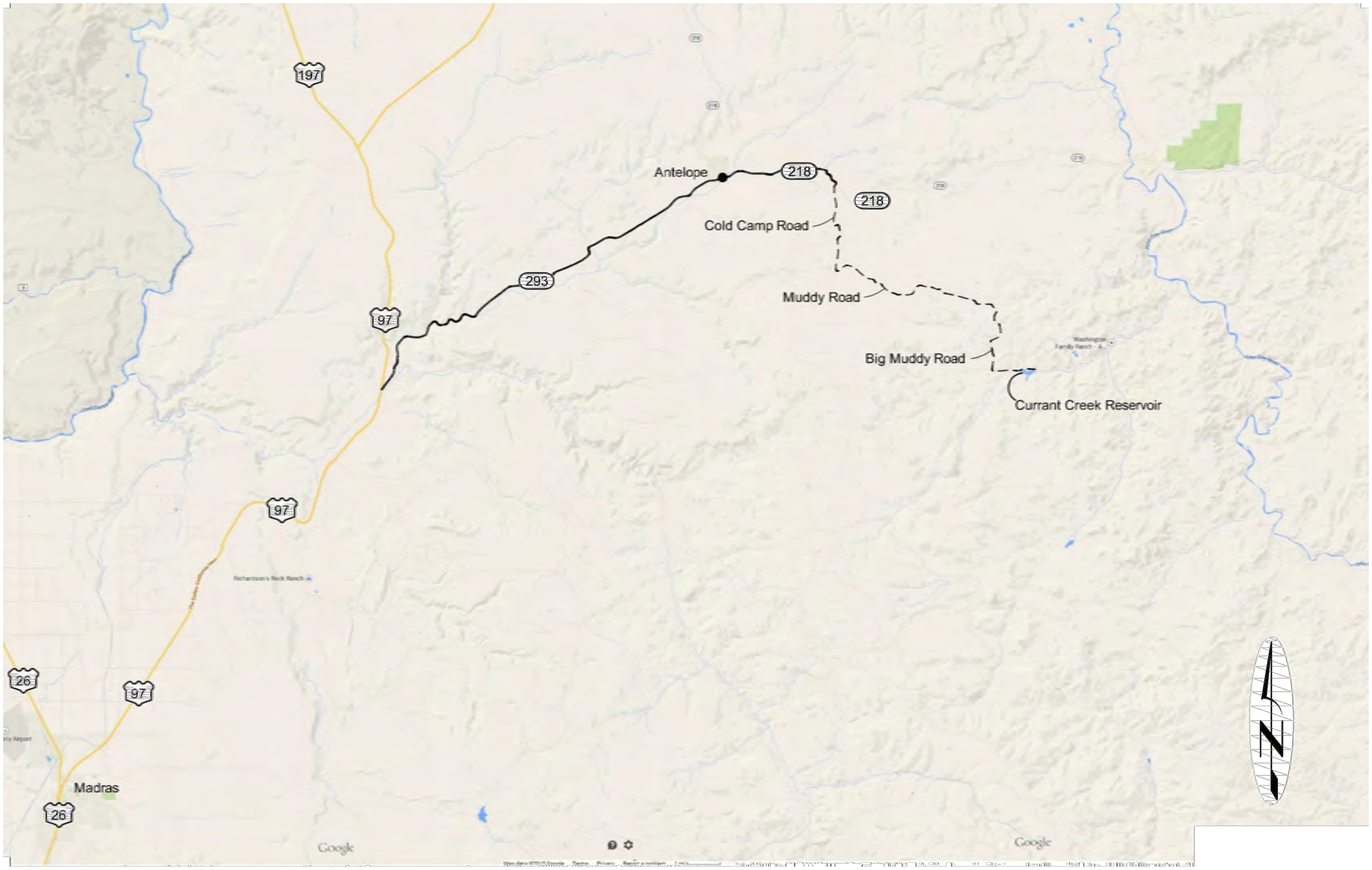
Borrow material is needed to raise the existing dam. Material quantities must be sufficient to raise the top of the existing dam by 5 feet by extending upward the 3 to 1 upstream slope of the dam to new top of the dam with a 20 foot width, then sloping downward at 1.5 to 1 to a catch point for the toe of the fill. The volume of fill could be 5,000 to 6,000 cubic yards (compacted).

Rip rap is needed for the upstream slope of the dam, including repair of parts of the slope and protection of the new slope from the dam raise. Rip rap is also needed for erosion protection in the new modified spillway channel. Although the volume is unknown at this time, there is likely rock available in the proposed excavation for rip rap purposes. Screening and sizing will be required to provide adequate sizing and a well-graded mix.

Borrow Area Locations

Material for embankment to raise the dam will be obtained from material excavated to construct the modified spillway. Some screening may be required to remove oversize rock debris and provide a suitable material for placement and compaction for raising the dam.

Rip rap will be generated from excavation to construct the modified spillway. Rip rap will also be generated from existing gravel to large boulder size material in the existing spillway channel.



0' 3 6
SCALE: 1" = 3 Miles

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H.A. McCOY
ENGINEERING & SURVEYING, LLC
1180 SW Lake Road, Suite 201 Redmond, OR 97756
Hayes McCoy
PE, PLS, CWRE
541-923-7554
hayes@ham-engr.com

PROJECT:
CURRENT CREEK DAM

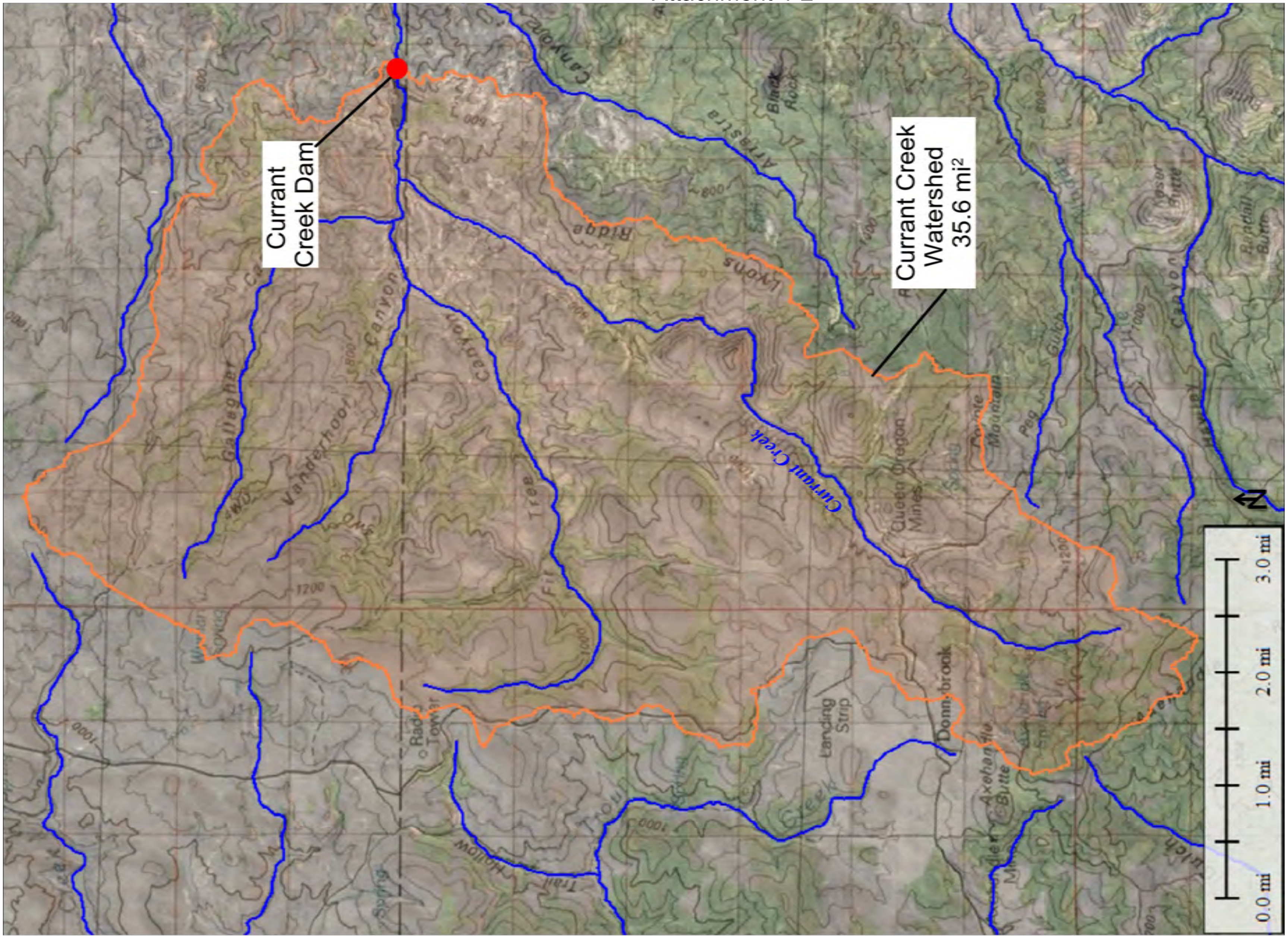
PROJECT LOCATION:
WASCO COUNTY, OREGON

CLIENT:
YOUNG LIFE

SHEET TITLE:
GENERAL LOCATION MAP

JOB NO. 1163.103
DRAWN BY: TDV/SCW
DRAWING:
FIG 1

FILE: \\freenas\NAS\HAM Engr\1163.103 WFR Dam Raise & Spillway\Engineering\Geotech Figures\200519-1163-GEO-FIGURES 1,4,6,7,9,11-15.dwg 5/27/2020 12:47 PM - Owner



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1180 SW Lake Road, Suite 201 Redmond, OR 97756

Hayes McCoy

PE, PLS, CWRE

541-923-7554 hayes@ham-engr.com

PROJECT: CURRENT CREEK DAM

PROJECT LOCATION: WASCO COUNTY, OREGON

CLIENT: YOUNG LIFE

SHEET TITLE: WATERSHED AREA

JOB NO. 1163.103

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DRAWING: FIG 2

FILE: \\freenas\NAS\HAM Engr\1163.103 WFR Dam Raise & Spillway\Engineering\Geotech Figures\200519-1163-GEO-FIGURES 1,4,6,7,9,11-15.dwg 5/27/2020 12:47 PM - Owner

FILE: \\freenas\NAS\HAM Engr\1163.103 WFR Dam Raise & Spillway\Engineering\Geotech Figures\200519-1163-GEO-FIGURES 1,4,6,7,9,11-15.dwg 5/27/2020 12:47 PM - Owner

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H.A. McCOY
ENGINEERING & SURVEYING, LLC
1180 SW Lake Road, Suite 201 Redmond, OR 97756
Hayes McCoy
PE, PLS, CWRE
541-923-7554
hayes@ham-engr.com

PROJECT:
CURRENT CREEK DAM

PROJECT LOCATION:
WASCO COUNTY, OR

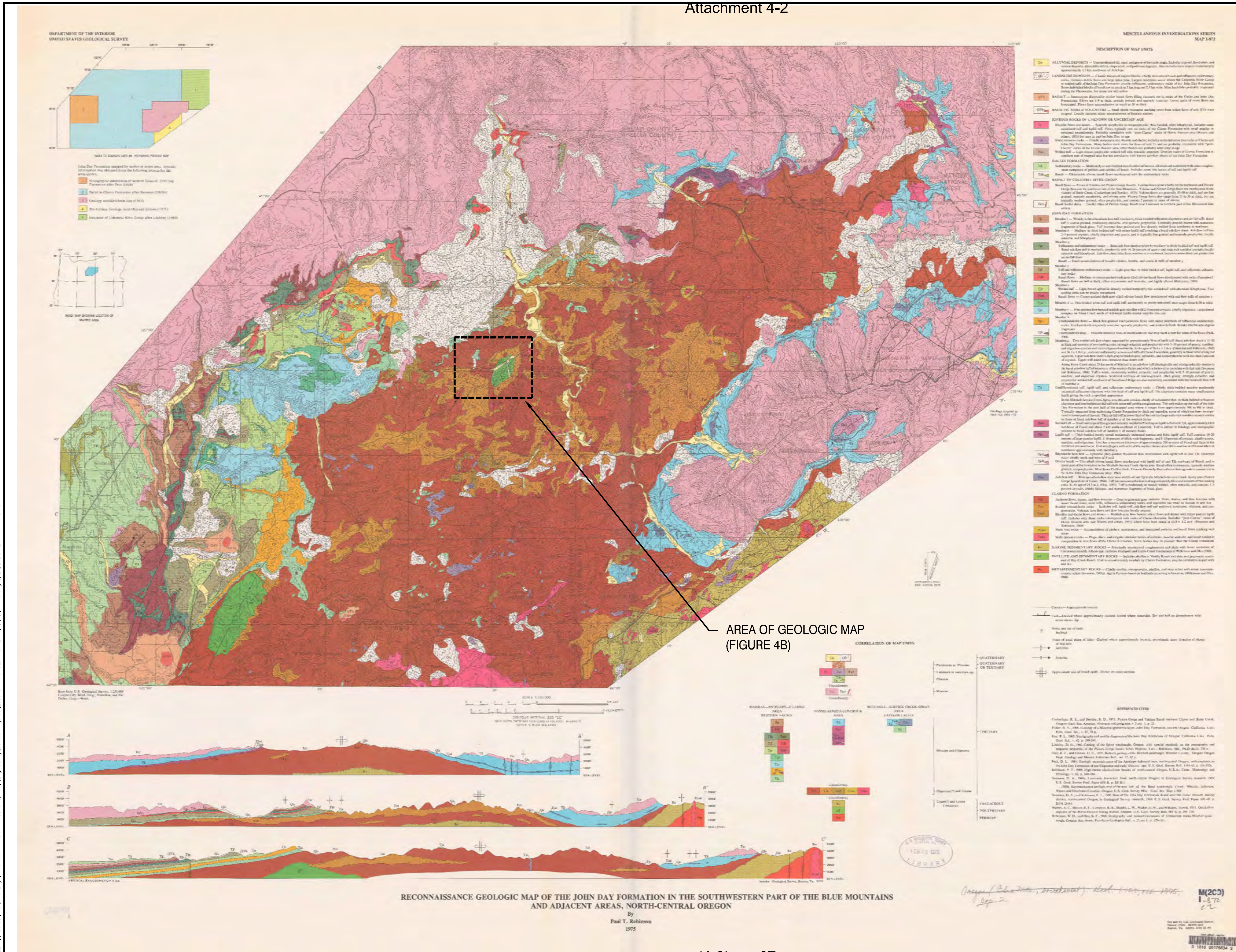
CLIENT:
YOUNG LIFE

SHEET TITLE:
RELATIONSHIP OF DAM AND RESERVOIR TO DOWNSTREAM INFRASTRUCTURE AND JOHN DAY RIVER

JOB NO. 1163.103

DRAWN BY: SCW

DRAWING:
FIG 3



FILE: \\freenas\NAS\HAM Engr\1163.103 WFR Dam Raise & Spillway\Engineering\Geotech Figures\200519-1163-GEO-FIGURES 1,4,6,7,9,11-15.dwg 5/27/2020 12:47 PM - Owner

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ENGINEERING & SURVEYING, LLC
1180 SW Lake Road, Suite 201, Redmond, OR 97756
Hayes McCoy
PE, PLS, CWRE
541-923-7554 hayes@ham-engr.com

PROJECT:
CURRENT CREEK DAM

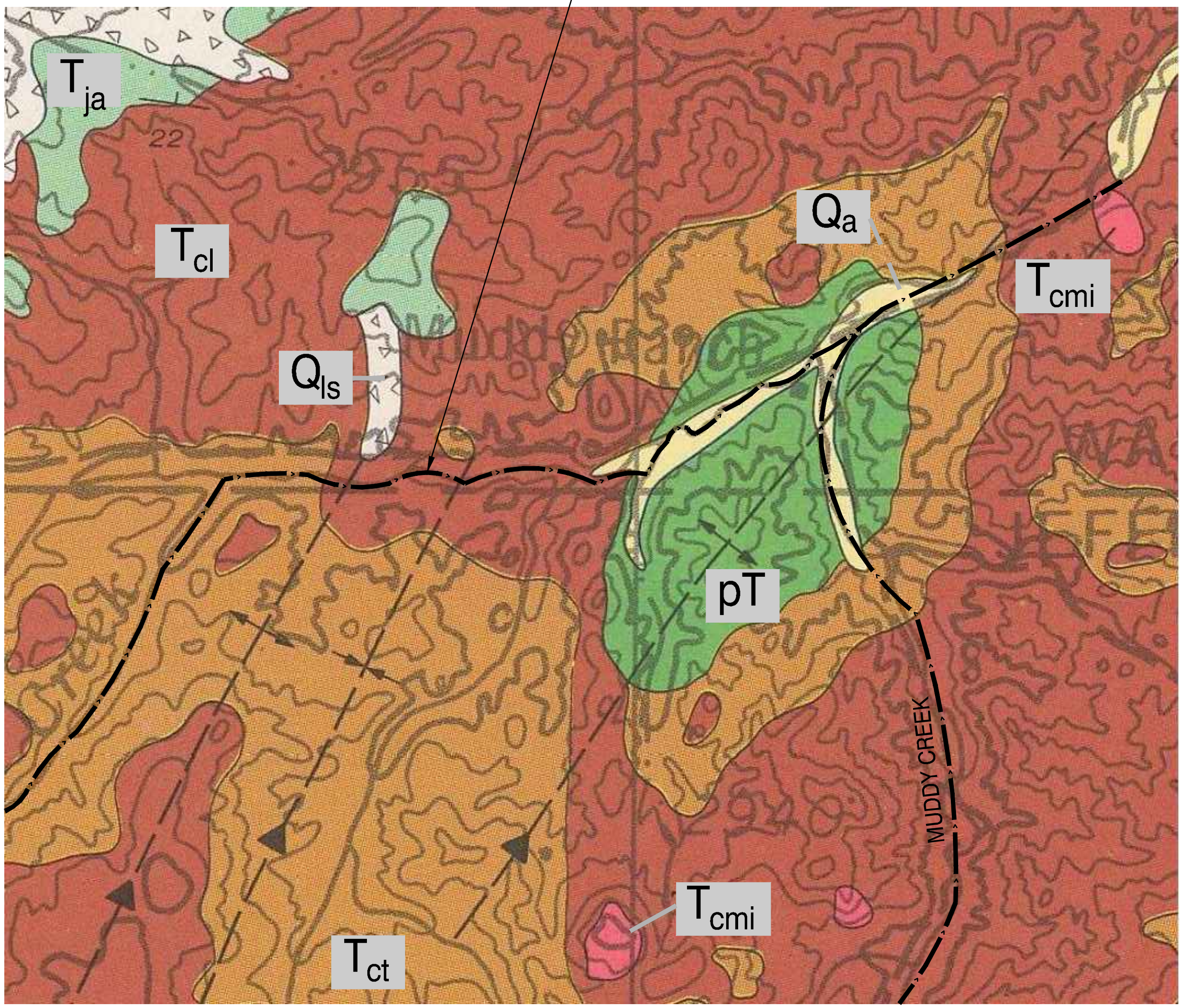
PROJECT LOCATION:
WASCO COUNTY, OR

CLIENT:
YOUNG LIFE

SHEET TITLE:
REGIONAL
GEOLOGIC MAP

JOB NO. 1163.103
DRAWN BY: SCW
DRAWING:
FIG 4A

CURRENT CREEK DAM (APPROX)



SOURCE

RECONNAISSANCE GEOLOGIC MAP OF THE JOHN DAY FORMATION, ROBINSON, P.T

USGS MISCELLANEOUS INVESTICATIONS SERIES MAP 1-872, 1975

EXPLANATION

Q_a	ALLUVIAL DEPOSITS
Q_{ls}	LANDSLIDE DEPOSITS, MOST OF PLEISTOCENE
T_{ja}	JOHN DAY FORMATION. MEMBER A. WELDED ASH FLOW TUFF SHEETS, LAPILLI TUFF
T_{cl}	CLARNO FORMATION. ANDESITIC FLOWS, DOMAS AND FLOW BRECCIAS
T_{ct}	CLARNO FORMATION. BEDDED VOLCANICLASTICS, TUFF, LAPILLI TUFF, SILTSTONE, CONGLOMERATE
T_{cmi}	CLARNO FORMATION. MAFIC INTRUSIVE ROCK
pT	CLARNO FORMATION. PHYLLITE AND SEDIMENTARY ROCKS

APPROX SCALE:

1" = 1000'

DATE: 5/28/20

DRAWING STATUS: ☒ GEOTECHNICAL REPORT

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Hayes McCoy PE, PLS, CWRE

hayes@ham-engr.com

PROJECT: CURRENT CREEK DAM

PROJECT LOCATION: WASCO COUNTY, OREGON

CLIENT: YOUNG LIFE

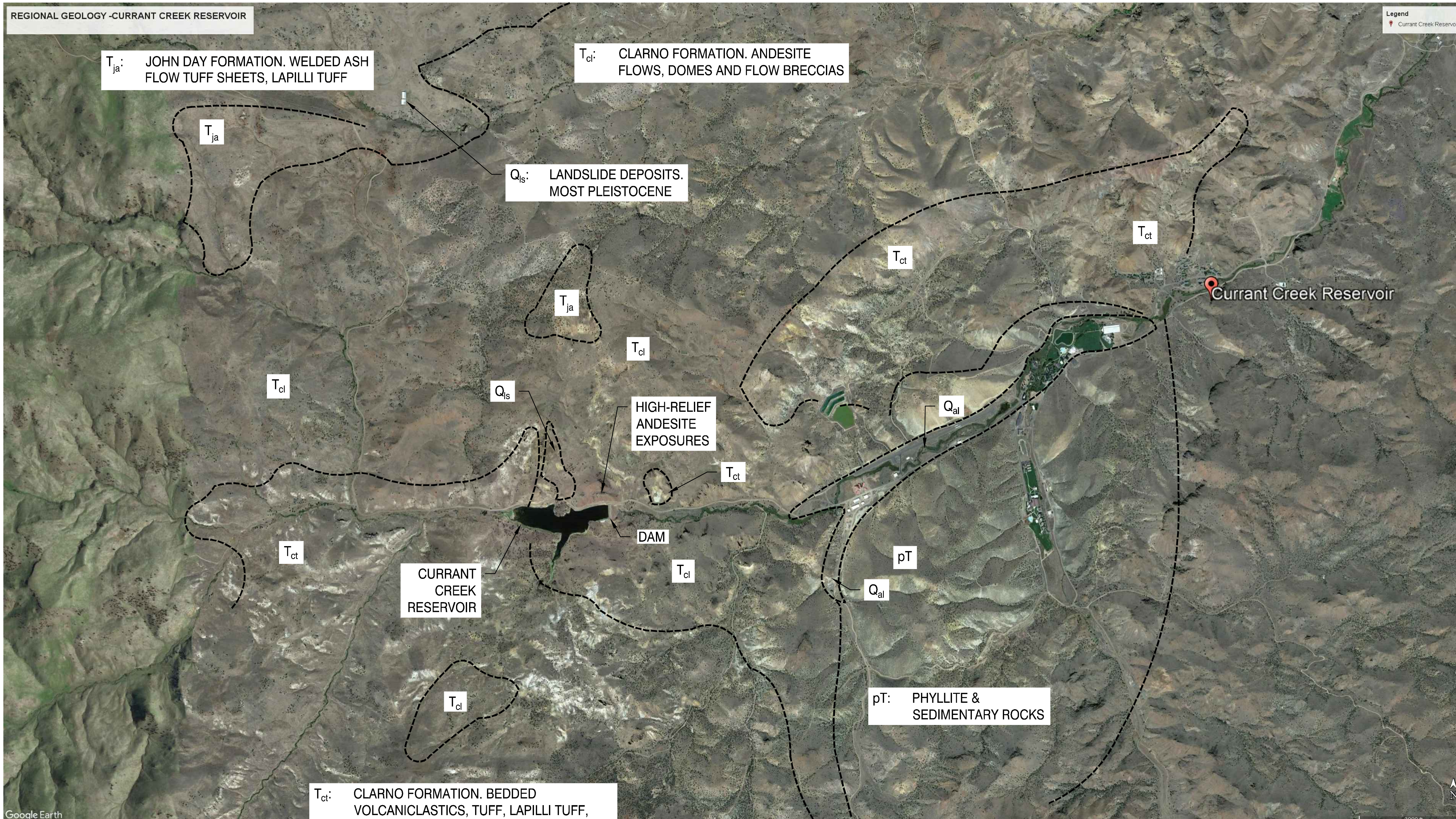
SHEET TITLE: REGIONAL GEOLOGIC MAP

JOB NO. 1163.103

DRAWN BY: TDV/SCW

DRAWING: FIG 4B

FILE: \\freenas\NAS\HAM Engr\1163.103 WFR Dam Raise & Spillway\Engineering\Geotech Figures\200519-1163-GEO-FIGURES 1,4,6,7,9,11-15.dwg 5/27/2020 12:47 PM - Owner



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No. REVISION: DATE:

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PROJECT: CURRENT CREEK DAM

PROJECT LOCATION: WASCO COUNTY, OR

CLIENT: YOUNG LIFE

SHEET TITLE: GEOLOGY OF DAM AND SPILLWAY AREA

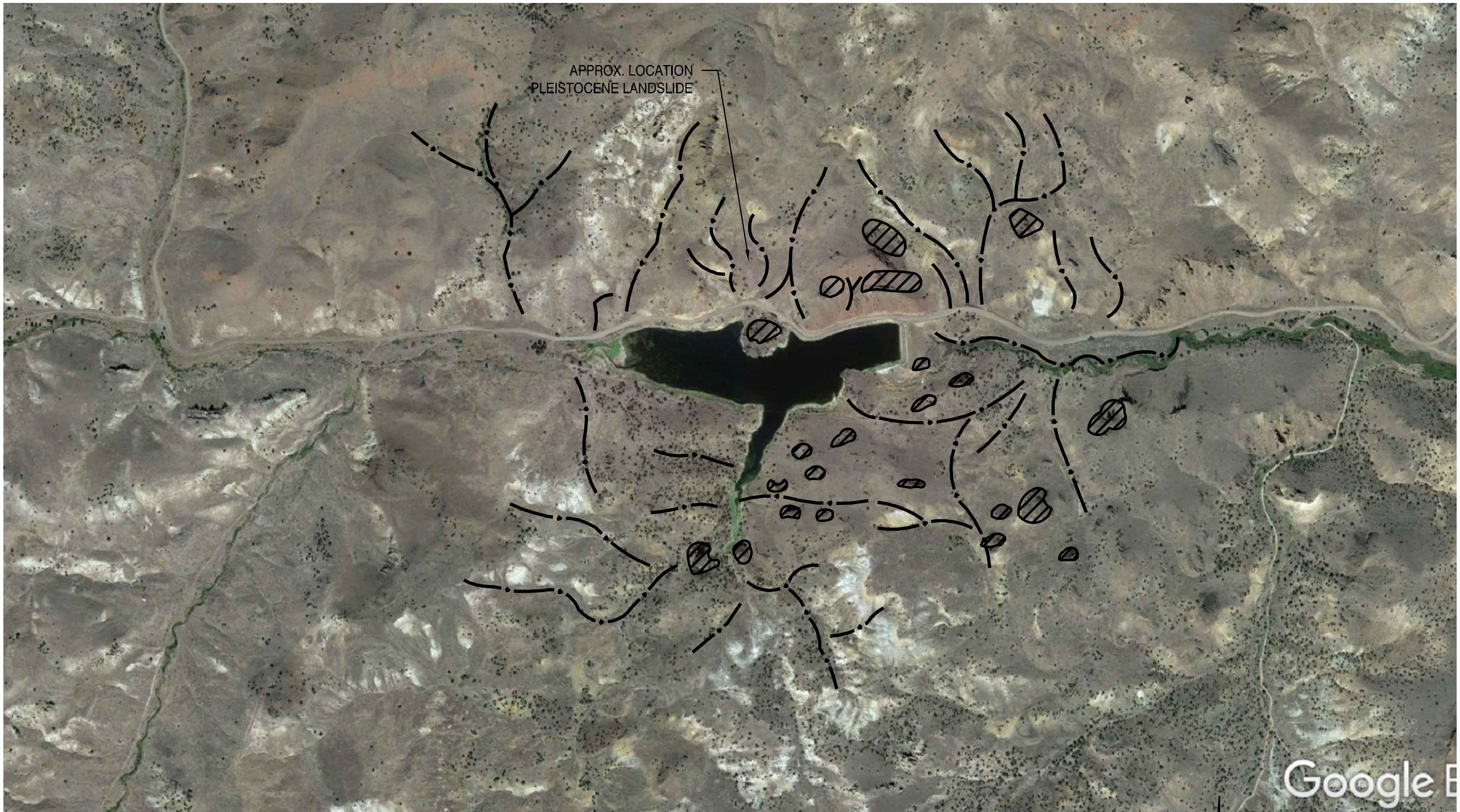
JOB NO. 1163.103

DRAWN BY: SCW

DRAWING: FIG 5

FILE: \\freenas\MAS\HAM Engr\1163.103 WFR Dam Raise & Spillway\Engineering\Geotech Figures\200519-1163-GEO-FIGURES 1,4,6,7,9,11-15.dwg 5/27/2020 12:47 PM - Owner

GENERAL GEOMORPHIC FEATURES OF DAM AND RESERVOIR AREA



EXPLANATION

MAJOR DRAINAGES

HIGH-RELIEF POINTS DUE TO EROSION-RESISTANT ANDESITE & ANDESITE BRECCIA

APPROX SCALE:
1" = 1600'

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ENGINEERING & SURVEYING, LLC
1180 SW Lake Road, Suite 201 Redmond, OR 97756
Hayes McCoy
PE, PLS, CWRE
541-923-7554
hayes@ham-engr.com

PROJECT: CURRENT CREEK DAM

PROJECT LOCATION: WASCO COUNTY, OREGON

CLIENT: YOUNG LIFE

SHEET TITLE: GEOMORPHIC CONDITIONS

JOB NO. 1163.103
DRAWN BY: TDV/SCW
DRAWING: FIG 6

FILE: \\freenos\NAS\HAM Engr\1163.103 WFR Dam Raise & Spillway\Engineering\Geotech Figures\200519-1163-GEO-FIGURES 1,4,6,7,9,11-15.dwg 5/27/2020 12:47 PM - Owner

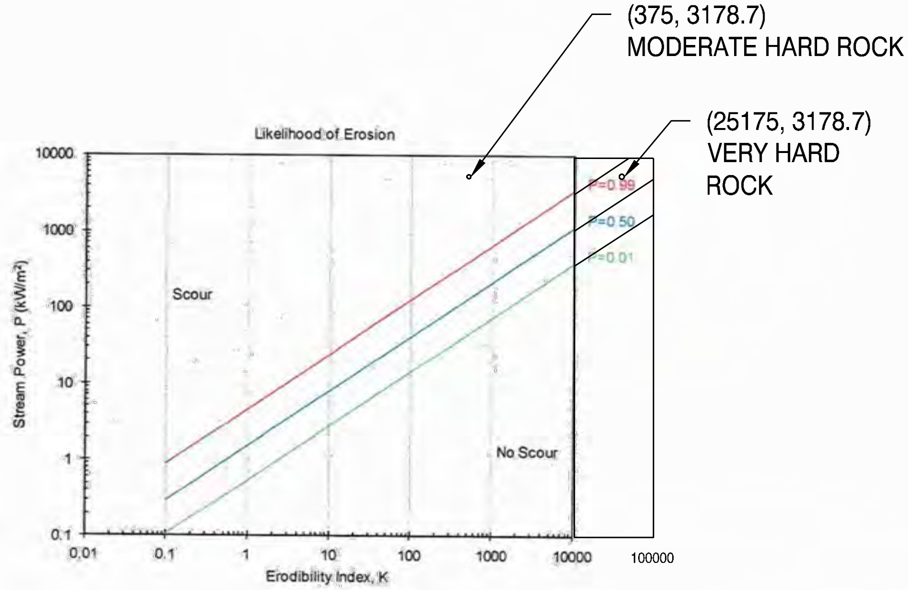

Figure IV-1-7 – Erodibility Threshold Probability Graph (Wibowo et al, 2005)

Judgment is required when applying these methods. The results can be sensitive to K_b, which is somewhat difficult to assess. In addition, materials will be more easily eroded on an abutment slope where there are more degrees of freedom for movement than in the bottom of a plunge pool where only the top of rock blocks are exposed. Cross jointing, if not present, can also increase the erosion resistance of the rock. These issues are not directly accounted for in these methods. Key block theory can be helpful in these situations to identify whether there are potentially removable blocks. A combination of the erodibility threshold graphs produced by Annandale and Wibowo can assist in providing a range when analyzing the likelihood of progression for embankment and spillway headcuts. See Figure IV-1-8 for a combined plot of the two methods.

REFERENCE INFORMATION:

1. "IV-1 EROSION OF ROCKS AND SOIL", U.S. ARMY CORPS OF ENGINEERS AND THE U.S. BUREAU OF RECLAMATION

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DATE: 5/28/20

H.A. McCOY
ENGINEERING & SURVEYING, LLC
1180 SW Lake Road, Suite 201 Redmond, OR 97756
Hayes McCoy
PE, PLS, CWRE
541-923-7554
hayes@ham-engr.com

PROJECT: CURRENT CREEK DAM

PROJECT LOCATION: WASCO COUNTY, OREGON

CLIENT: YOUNG LIFE

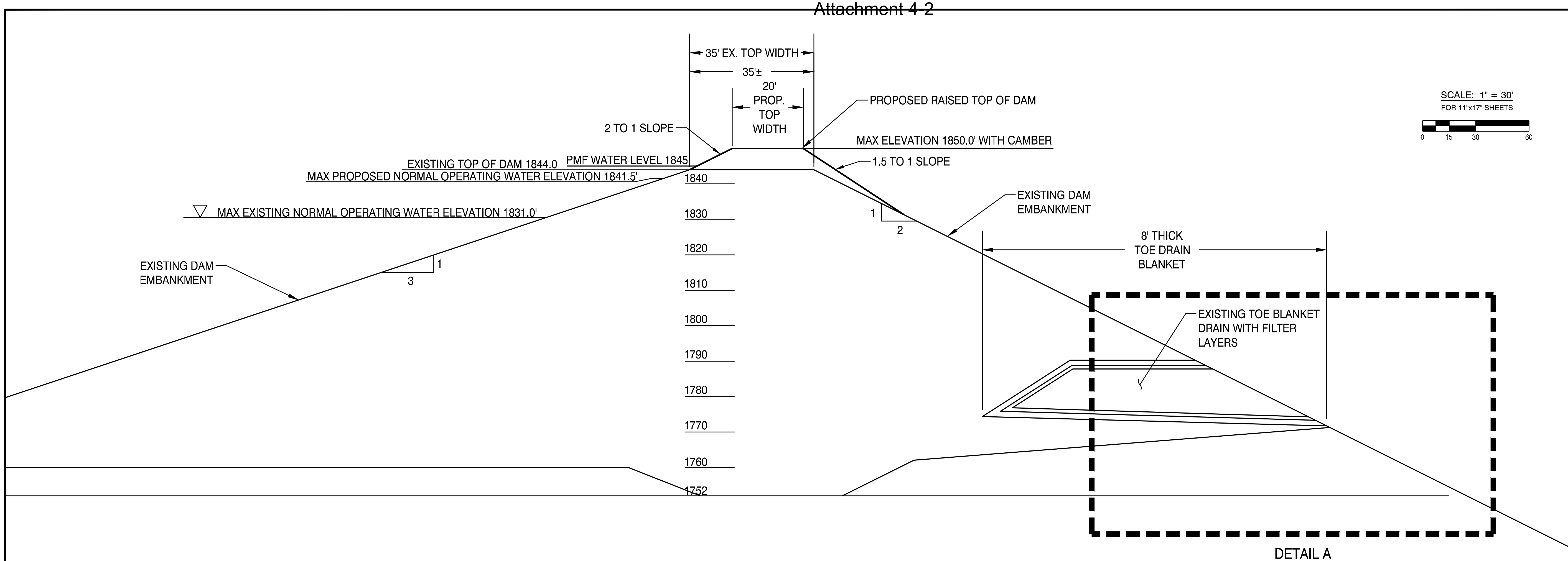
SHEET TITLE: ERODIBILITY THRESHOLD PROBABILITY GRAPH

JOB NO. 1163.103

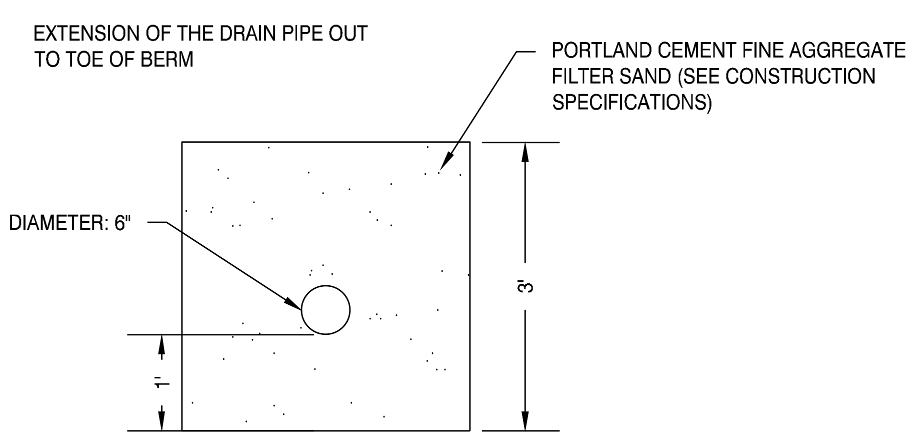
DRAWN BY: TDV/SCW

DRAWING: FIG 7

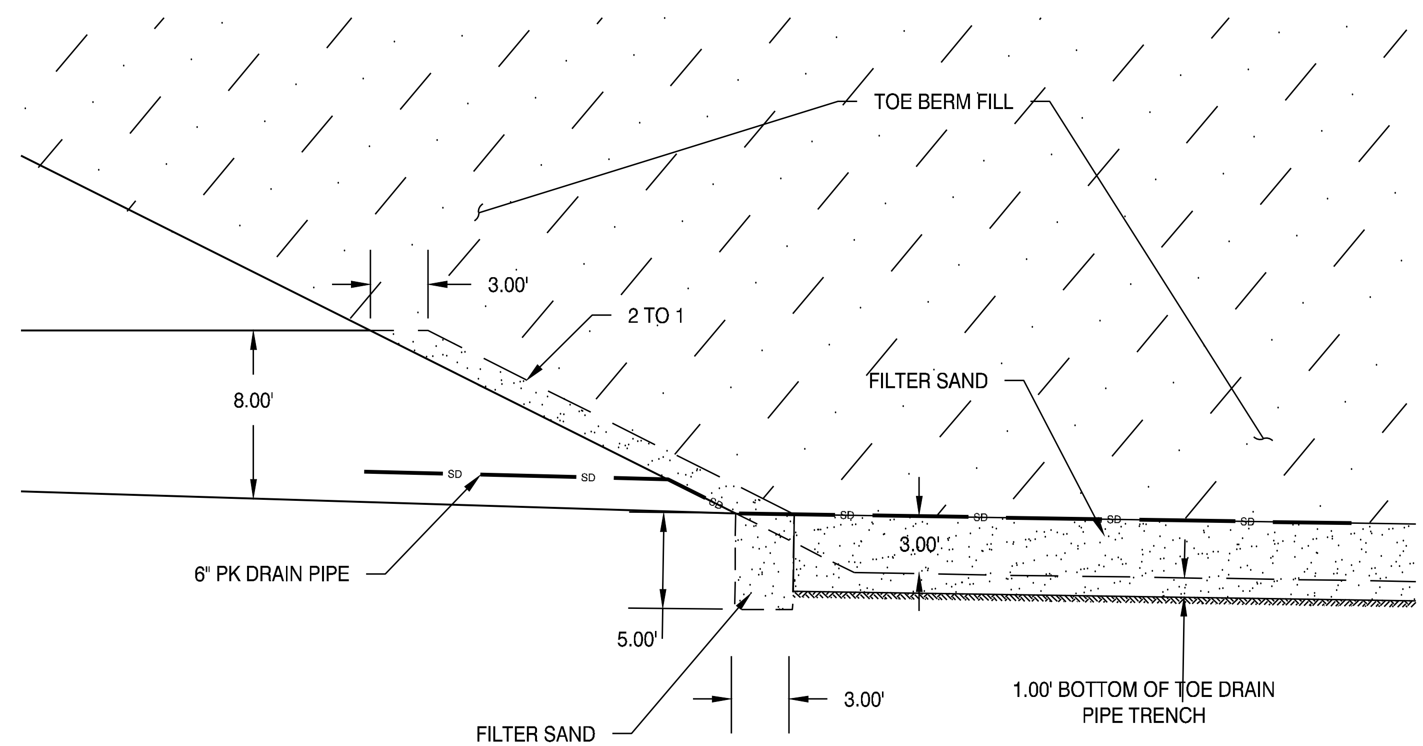
FILE: \\freenas\NAS\HAM Engr\1163.103 WFR Dam Raise & Spillway\Engineering\Geotech Figures\200519-1163-GEO-FIGURES 1,4,6,7,9,11-15.dwg 5/27/2020 12:47 PM - Owner



CROSS SECTION - EXISTING DAM & PROPOSED DAM



SCALE: 1" = 2'



SCALE: 1" = 10'

DETAIL A

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H.A. McCOY ENGINEERING & SURVEYING, LLC

PROJECT: CURRENT CREEK DAM

PROJECT LOCATION: WASCO COUNTY, OR

CLIENT: YOUNG LIFE

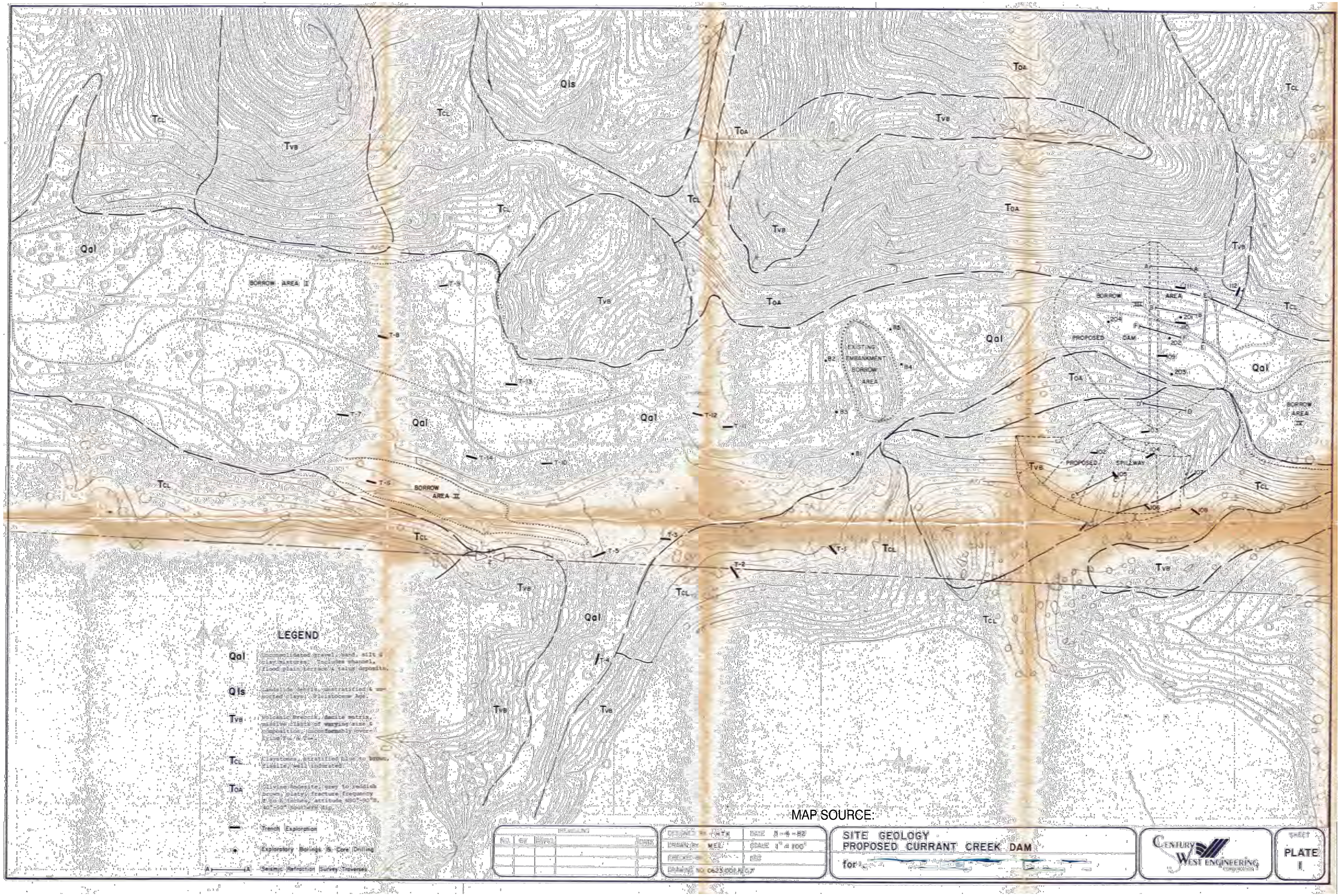
SHEET TITLE: CROSS SECTION OF EXISTING DAM

JOB NO. 1163.103

DRAWN BY: SCW

DRAWING: FIG 8

FILE: \\freenas\MAS\HAM Engr\1163.103 WFR Dam Raise & Spillway\Engineering\Geotech Figures\200519-1163-GEO-FIGURES 1,4,6,7,9,11-15.dwg 5/27/2020 12:47 PM - Owner



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ENGINEERING & SURVEYING, LLC

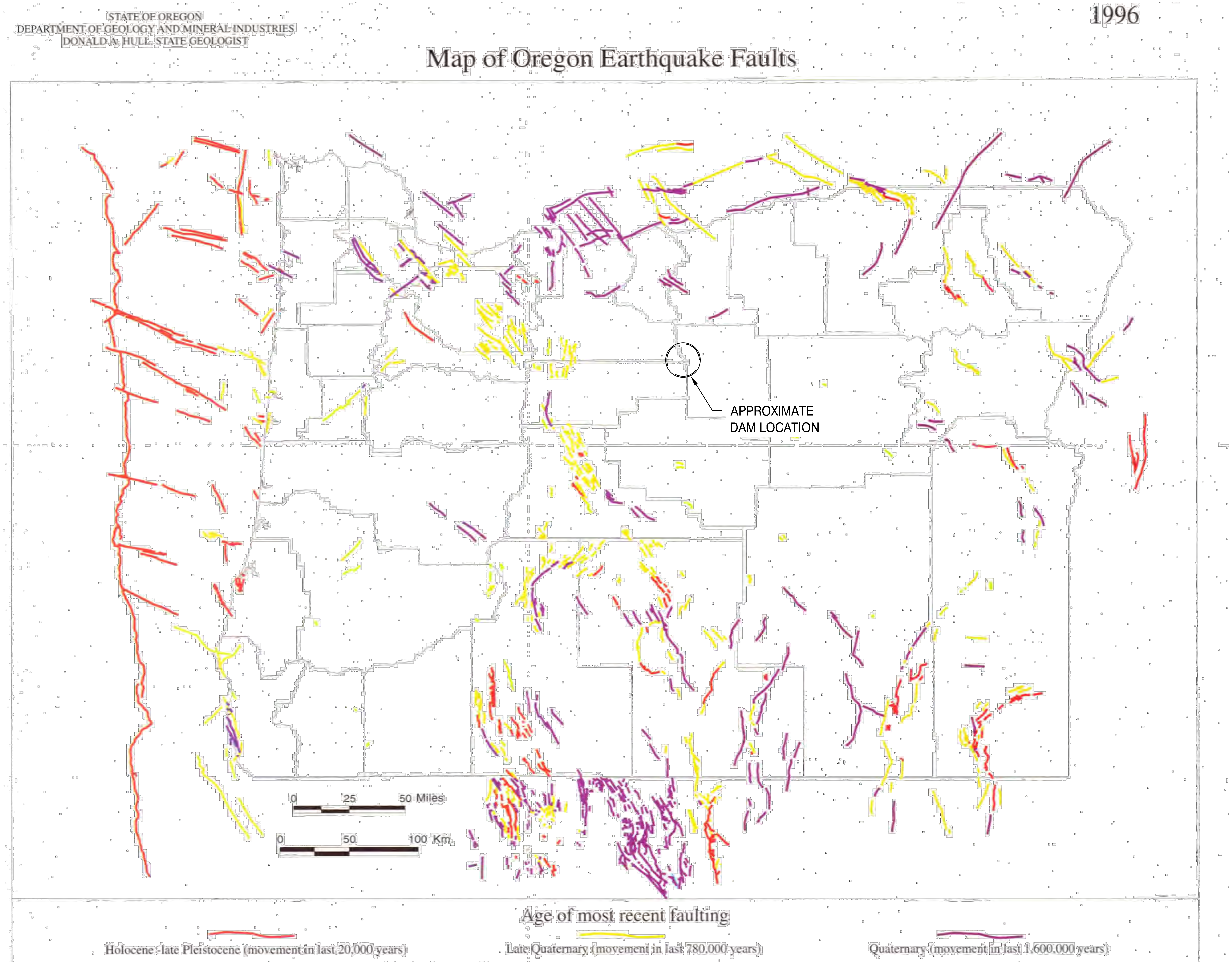
PROJECT:
CURRENT CREEK DAM

PROJECT LOCATION:
WASCO COUNTY, OR

CLIENT:
YOUNG LIFE

SHEET TITLE:
LOCATION MAP OF BORINGS, EXPLORATORY BACKHOE PITS & SEISMIC PROFILES

JOB NO. 1163.103
DRAWN BY: SCW
DRAWING:
FIG 9



FILE: \\freenas\NAS\HAM Engr\1163.103 WFR Dam Raise & Spillway\Engineering\Geotech Figures\200519-1163-GEO-FIGURES 1,4,6,7,9,11-15.dwg 5/27/2020 12:47 PM - Owner

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H.A. McCOY ENGINEERING & SURVEYING, LLC

PROJECT: CURRENT CREEK DAM

PROJECT LOCATION: WASCO COUNTY, OR

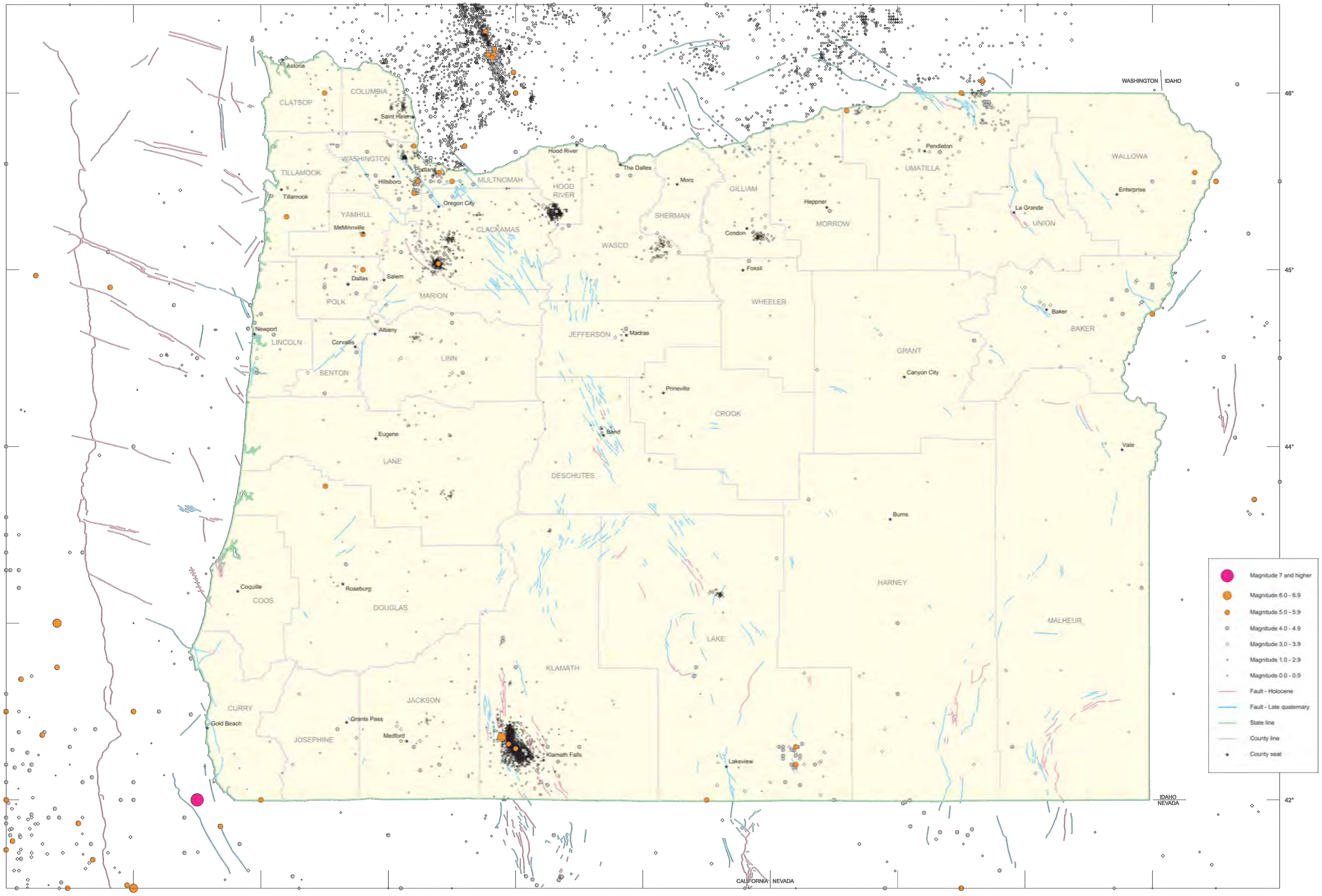
CLIENT: YOUNG LIFE

SHEET TITLE: DAM LOCATION RELATIVE TO MAPPED FAULTS

JOB NO. 1163.103

DRAWN BY: SCW

DRAWING: FIG 10



REFERENCE INFORMATION:

1. CLARK A. NIEWENDORP & MARK E. NEUHAUS, "MAP OF SELECTED EARTHQUAKES FOR OREGON, 1841-2002", STATE OF OREGON DEPARTMENT OF GEOLOGY AND MINERAL INDUSTRIES, 2003

FILE: \\freenas\NAS\HAM Engr\1163.103 WFR Dam Raise & Spillway\Engineering\Geotech Figures\200519-1163-GEO-FIGURES 1,4,6,7,9,11-15.dwg 5/27/2020 12:47 PM - Owner

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Hayes McCoy PE, PLS, CWRE

PROJECT: CURRENT CREEK DAM

PROJECT LOCATION: WASCO COUNTY, OR

CLIENT: YOUNG LIFE

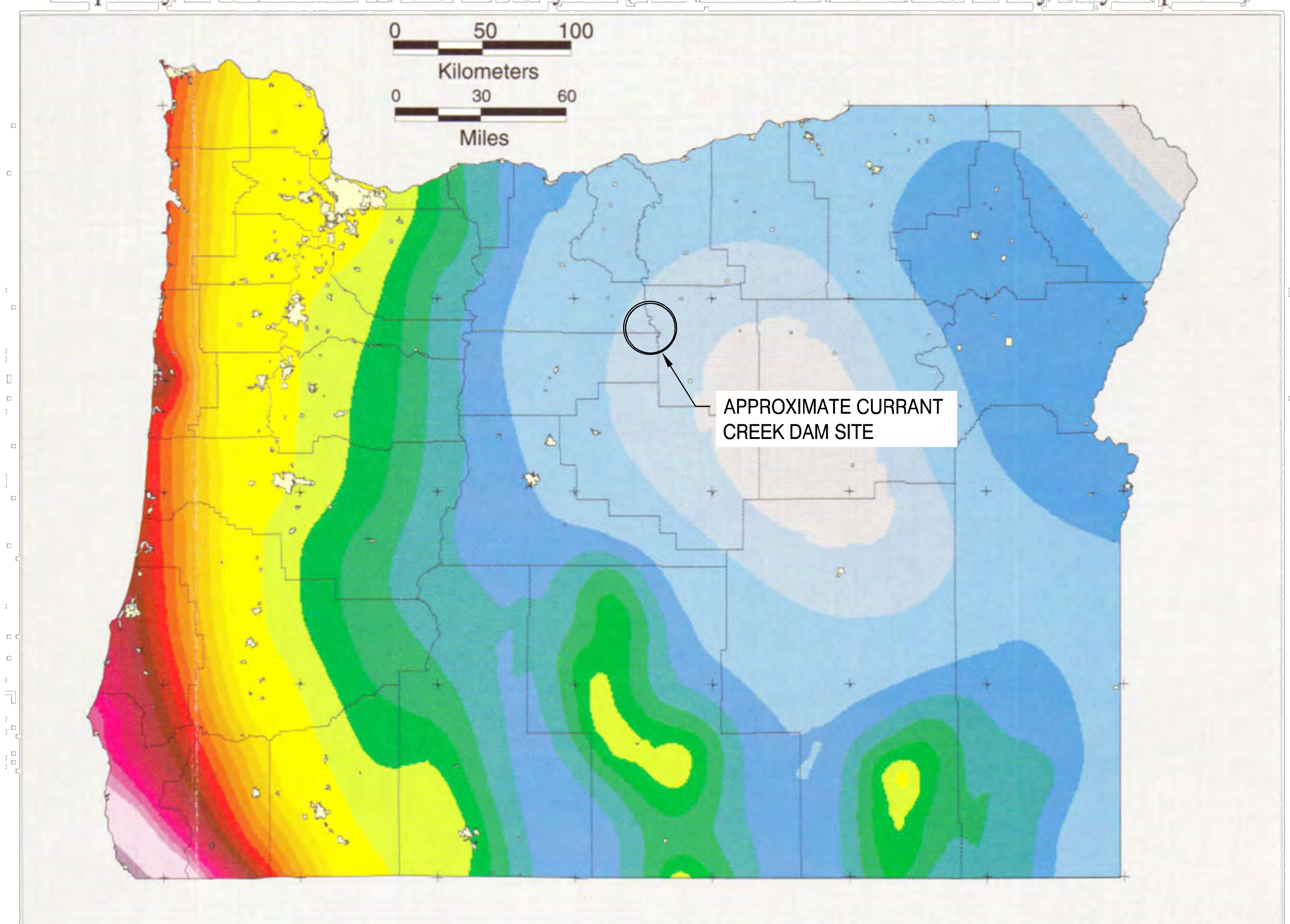
SHEET TITLE: DOGAMI EARTHQUAKE MAP- 1841-2002

JOB NO. 1163.103

DRAWN BY: SCW

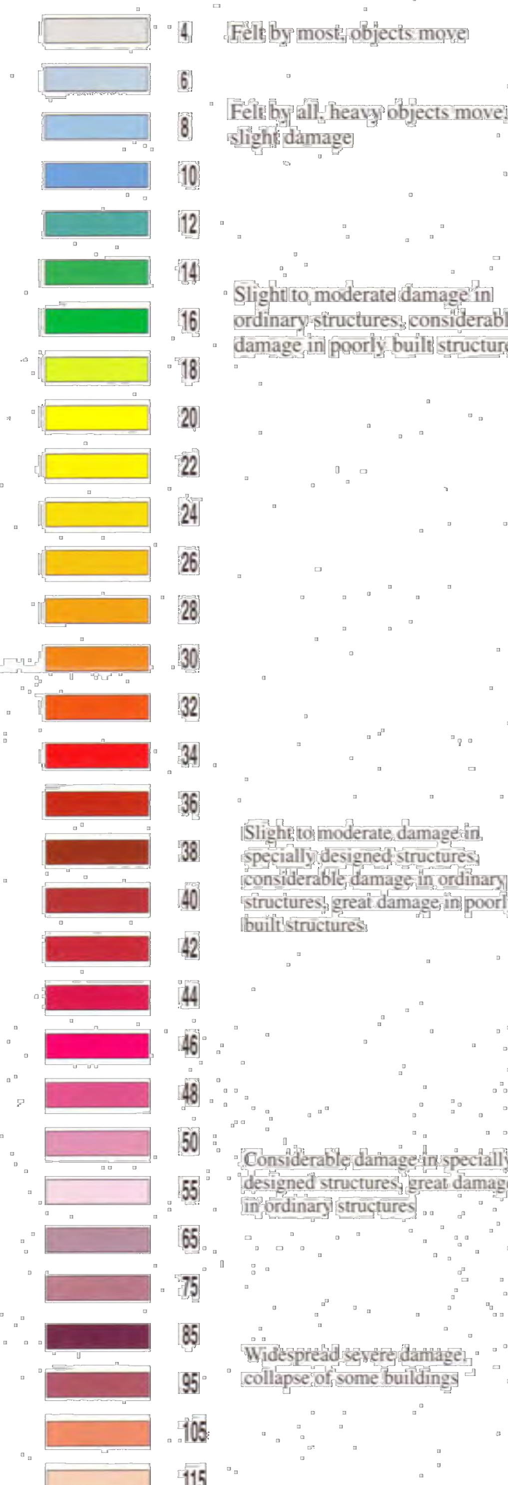
DRAWING: FIG 11

Map of Maximum Earthquake Shaking (Peak Ground Acceleration) expected in Oregon with a frequency of occurrence of once in 500 years (10% chance of occurrence in any 50 year period)



Ground Shaking Explanation for the 500, 1000 and 2500 year recurrence maps

Peak Ground Acceleration expressed as a percentage of gravity



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ENGINEERING & SURVEYING, LLC
1180 SW Lake Road, Suite 201 Redmond, OR 97756
Hayes McCoy
PE, PLS, CWRE
541-923-7554
hayes@ham-engr.com

PROJECT: CURRENT CREEK DAM
PROJECT LOCATION: WASCO COUNTY, OR
CLIENT: YOUNG LIFE

SHEET TITLE: 500-YEAR SHAKING MAP

JOB NO. 1163.103
DRAWN BY: SCW
DRAWING: FIG 12

FILE: \\freenas\NAS\HAM Engr\1163.103 WFR Dam Raise & Spillway\Engineering\Geotech Figures\200519-1163-GEO-FIGURES 1,4,6,7,9,11-15.dwg 5/27/2020 12:47 PM - Owner

FILE: \\freenas\NAS\HAM Engr\1163.103 WFR Dam Raise & Spillway\Engineering\Geotech Figures\200519-1163-GEO-FIGURES 1,4,6,7,9,11-15.dwg 5/27/2020 12:47 PM - Owner

-- Earthquake Planning Scenario --

ShakeMap for Casc9p0expanded Scenario

Scenario Date: Sep 20, 2011 06:00:00 AM MDT M 9.0 N45.73 W125.12 Depth: 20.0km

PLANNING SCENARIO ONLY -- Map Version 1 Processed 2017-01-30 03:41:06 PM MST

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	<0.02	0.1	1.4	4.7	9.6	20	41	86	>178
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Scale based upon Worden et al. (2012)

JOB NO. 1163.103
DRAWN BY: SCW
DRAWING:
FIG 13

SHEET TITLE:
EPICENTER LOCATION CASCADIA SUBDUCTION ZONE

PROJECT:
CURRENT CREEK DAM

PROJECT LOCATION:
WASCO COUNTY, OR

CLIENT:
YOUNG LIFE

H.A. McCOY ENGINEERING & SURVEYING, LLC
1110 SW Lake Road, Suite 201, Redmond, OR 97756
Hayes McCoy
PE, PLS, CWRE
541-923-7554 hayes@ham-engr.com

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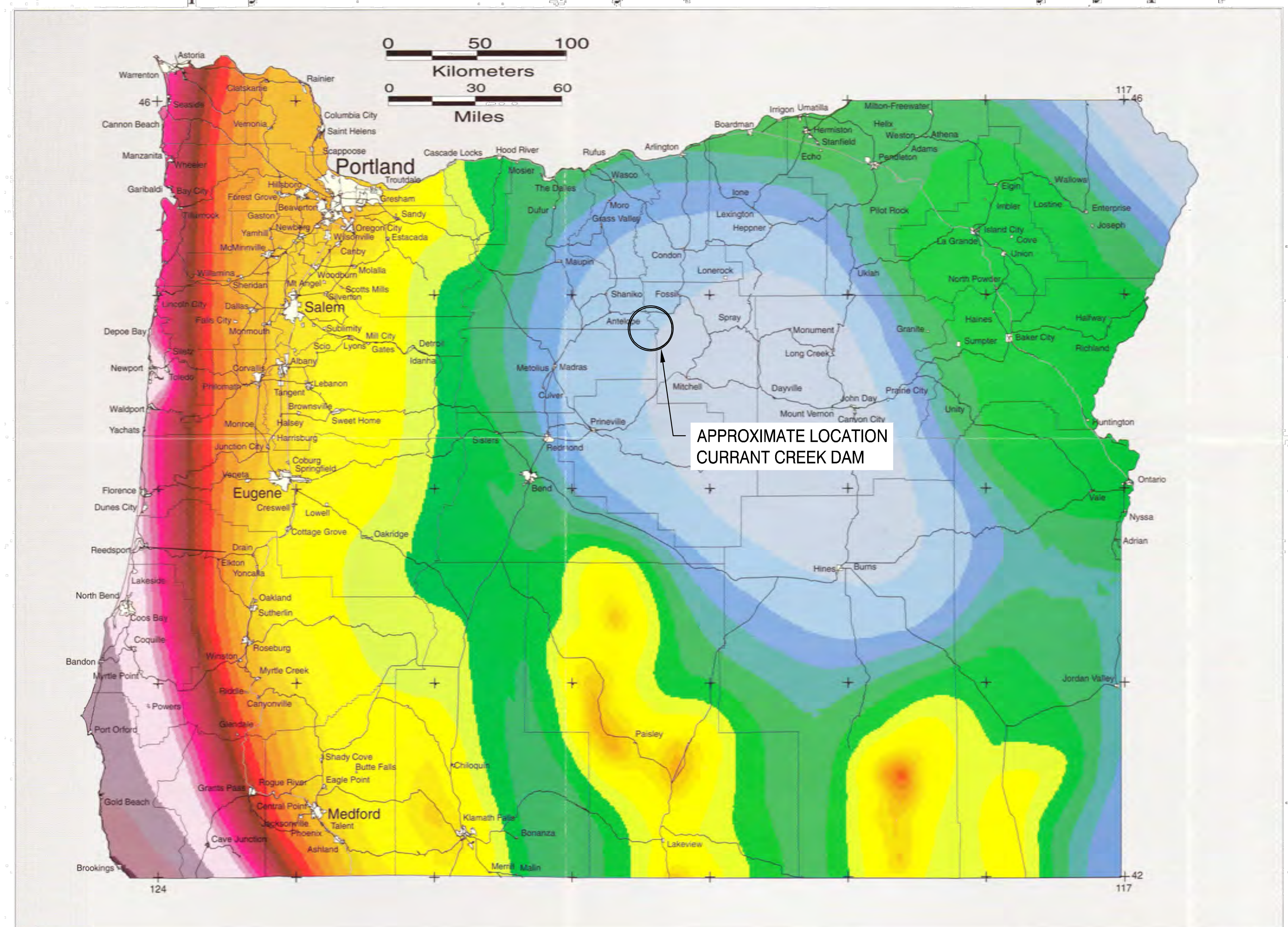
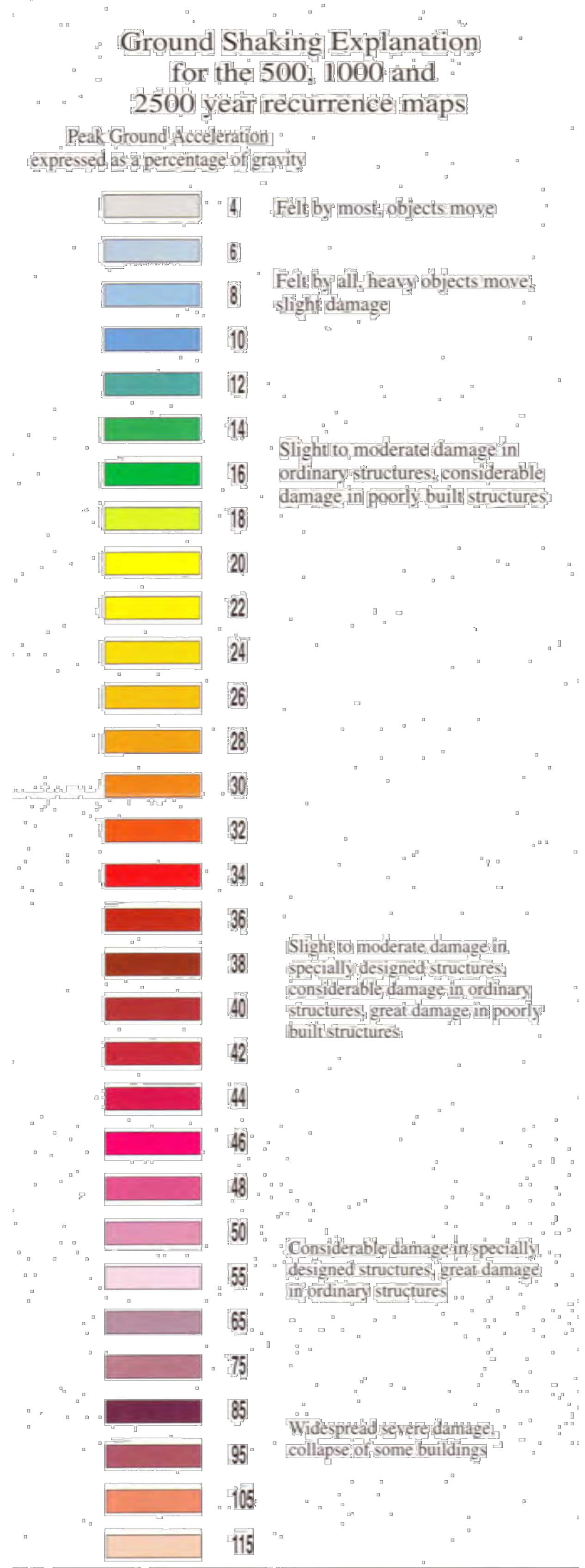
Map of Maximum Earthquake Shaking (Peak Ground Acceleration) expected in Oregon with a frequency of occurrence of once in 1000 years (5% chance of occurrence in any 50 year period)





DATE:

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H.A. McCOY ENGINEERING & SURVEYING, LLC

PROJECT: CURRENT CREEK DAM

PROJECT LOCATION: WASCO COUNTY, OR

CLIENT: YOUNG LIFE

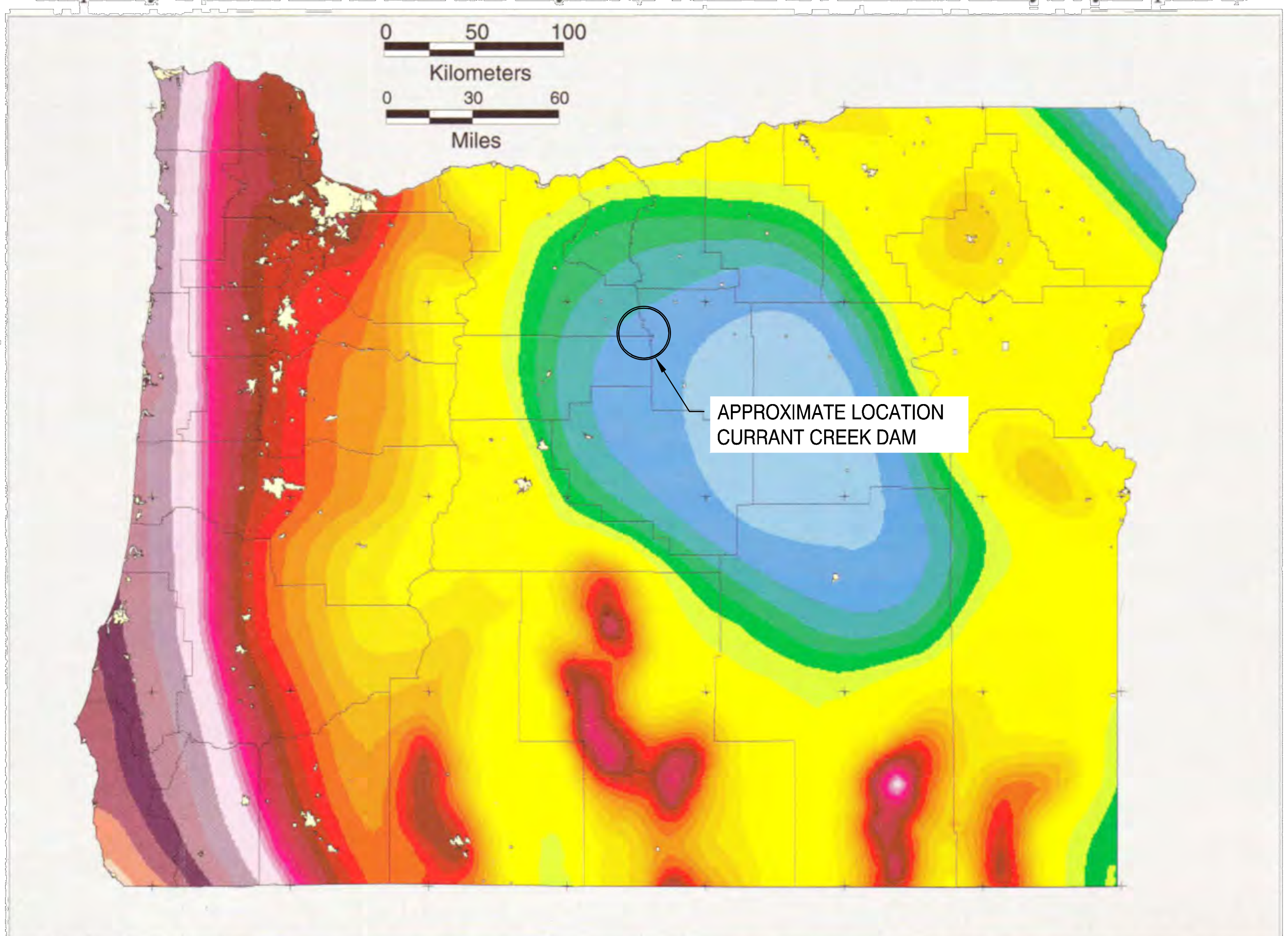
SHEET TITLE: 1,000-YEAR SHAKING MAP

JOB NO. 1163.103

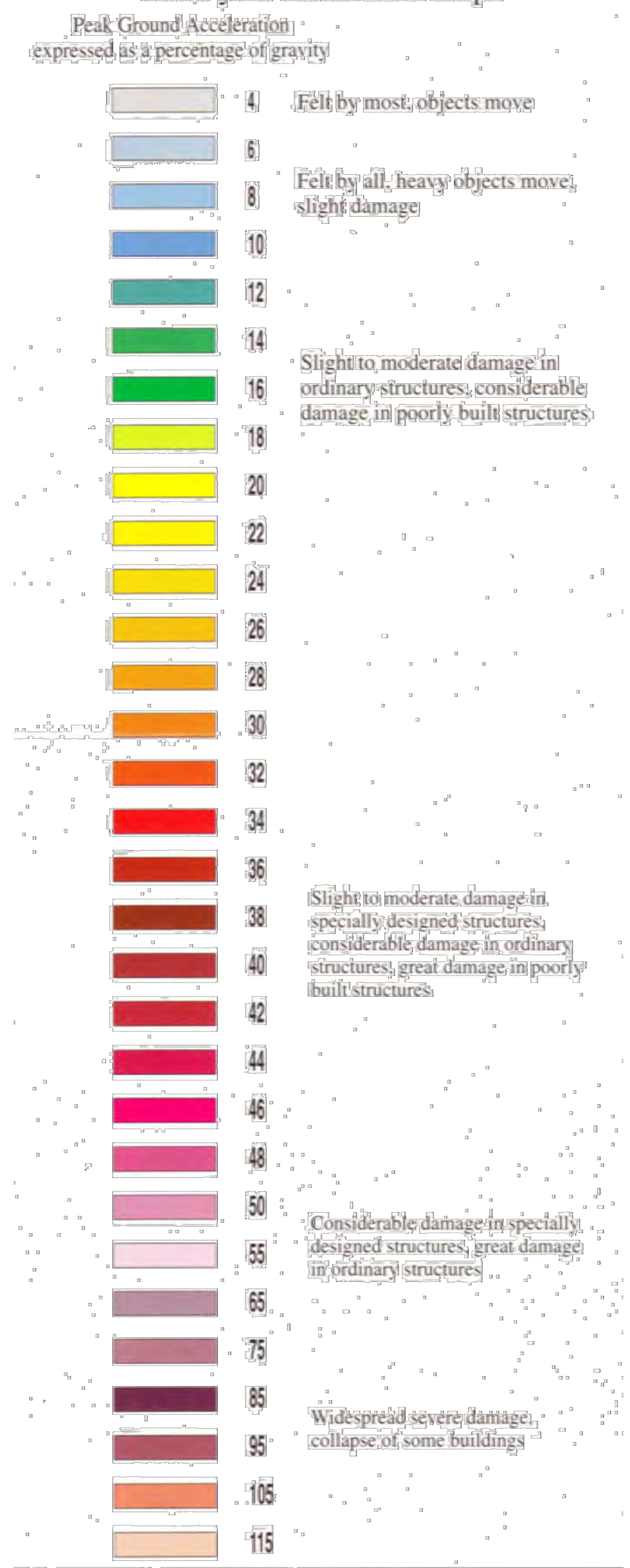
DRAWN BY: SCW

DRAWING: FIG 15

Map of Maximum Earthquake Shaking (Peak Ground Acceleration) expected in Oregon with a frequency of occurrence of once in 2500 years (2% chance of occurrence in any 50 year period)



Ground Shaking Explanation for the 500, 1000 and 2500 year recurrence maps



No.	REVISION:	DATE:

DATE: 5/19/20

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H.A. McCOY ENGINEERING & SURVEYING, LLC

PROJECT: CURRENT CREEK DAM

PROJECT LOCATION: WASCO COUNTY, OR

CLIENT: YOUNG LIFE

SHEET TITLE: 2,500-YEAR SHAKING MAP

JOB NO. 1163.103

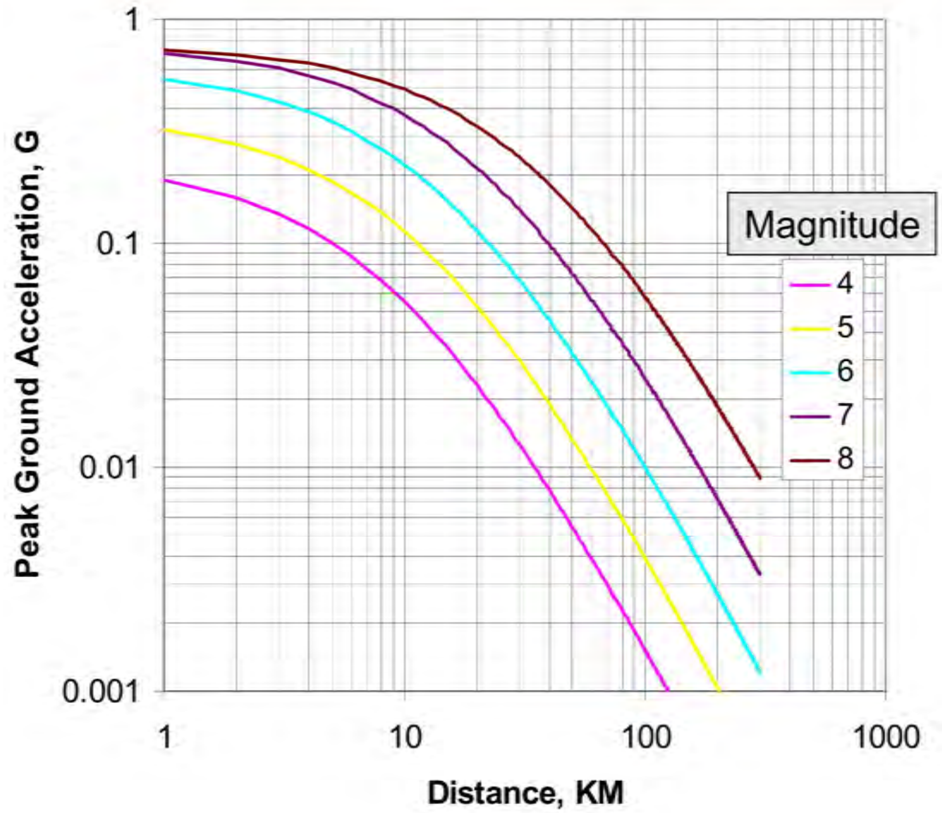
DRAWN BY: SCW

DRAWING: FIG 16

FILE: \\freenas\NAS\HAM Engr\1163.103 WFR Dam Raise & Spillway\Engineering\Geotech Figures\200519-1163-GEO-FIGURES 1,4,6,7,9,11-15.dwg 5/27/2020 12:47 PM - Owner

Attenuation Relation for Shallow Crustal Earthquakes

(Sadigh, Chang, Egan, Makdisi, and Youngs)



FEMA **Instructional Material Complementing *FEMA 451, Design Examples*** **Seismic Hazard Analysis 5a - 21**

REFERENCE INFORMATION:

1. SADIGH, CHANG, EGAN, MAKDISI AND YOUNGS, "FEMA 451B TOPIC 5A NOTES", FEDERAL EMERGENCY MANAGEMENT AGENCY

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DATE: 5/28/20

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PROJECT: CURRENT CREEK DAM

PROJECT LOCATION: WASCO COUNTY, OREGON

CLIENT: YOUNG LIFE

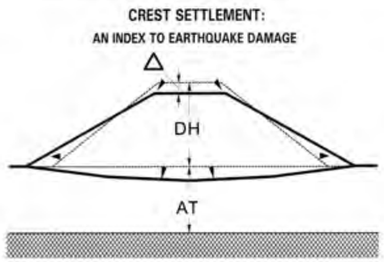
SHEET TITLE: ATTENUATION RELATION FOR SHALLOW CRUSTAL EARTHQUAKES

JOB NO. 1163.103

DRAWN BY: TDV/SCW

DRAWING: FIG 17

FILE: \\freenas\NAS\HAM Engr\1163.103 WFR Dam Raise & Spillway\Engineering\Geotech Figures\200519-1163-GEO-FIGURES 1,4,6,7,9,11-15.dwg 5/27/2020 12:47 PM - Owner



$$\% \text{ STTLMT} = \frac{\Delta}{DH + AT} \times 100$$

·F DAM TO ROCK COR TRENCH
LLUVIAL THICKNESS UNDER DAM FLANKS

.MT x (DH + AT)
03 x (28.2m+4.5m)
cm

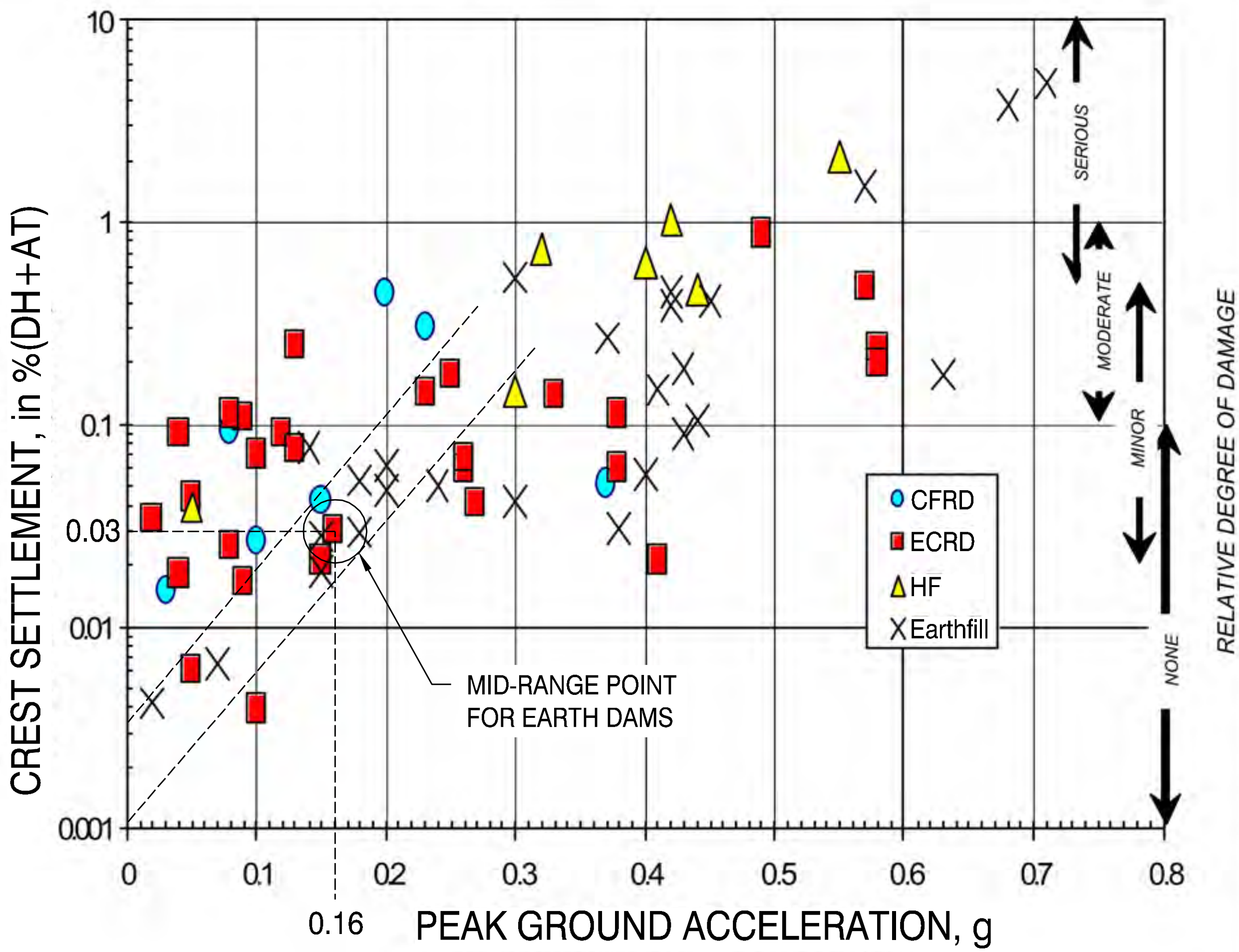


FIGURE 1. SETTLEMENT OF EMBANKMENT DAMS DURING EARTHQUAKE

REFERENCE INFORMATION:

1. J.R. SWAISGOOD, "EMBANKMENT DAM DEFORMATIONS CAUSED BY EARTHQUAKES", 2003 PACIFIC CONFERENCE ON EARTHQUAKE ENGINEERING

DRAWING STATUS: ☒ GEOTECHNICAL REPORT | DATE: 5/28/20

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1180 SW Lake Road, Suite 201 Redmond, OR 97756
Hayes McCoy
PE, PLS, CWRE
541-923-7554
hayes@ham-engr.com

PROJECT:
CURRENT CREEK DAM

PROJECT LOCATION:
WASCO COUNTY, OREGON

CLIENT:
YOUNG LIFE

SHEET TITLE:
SWAISGOOD ANALYSIS - DAM DEFORMATION POTENTIAL

JOB NO. 1163.103
DRAWN BY: TDV/SCW
DRAWING:
FIG 18A

FILE: \\freenas\NAS\HAM Engr\1163.103 WFR Dam Raise & Spillway\Engineering\Geotech Figures\200519-1163-GEO-FIGURES 1,4,6,7,9,11-15.dwg 5/27/2020 12:47 PM - Owner

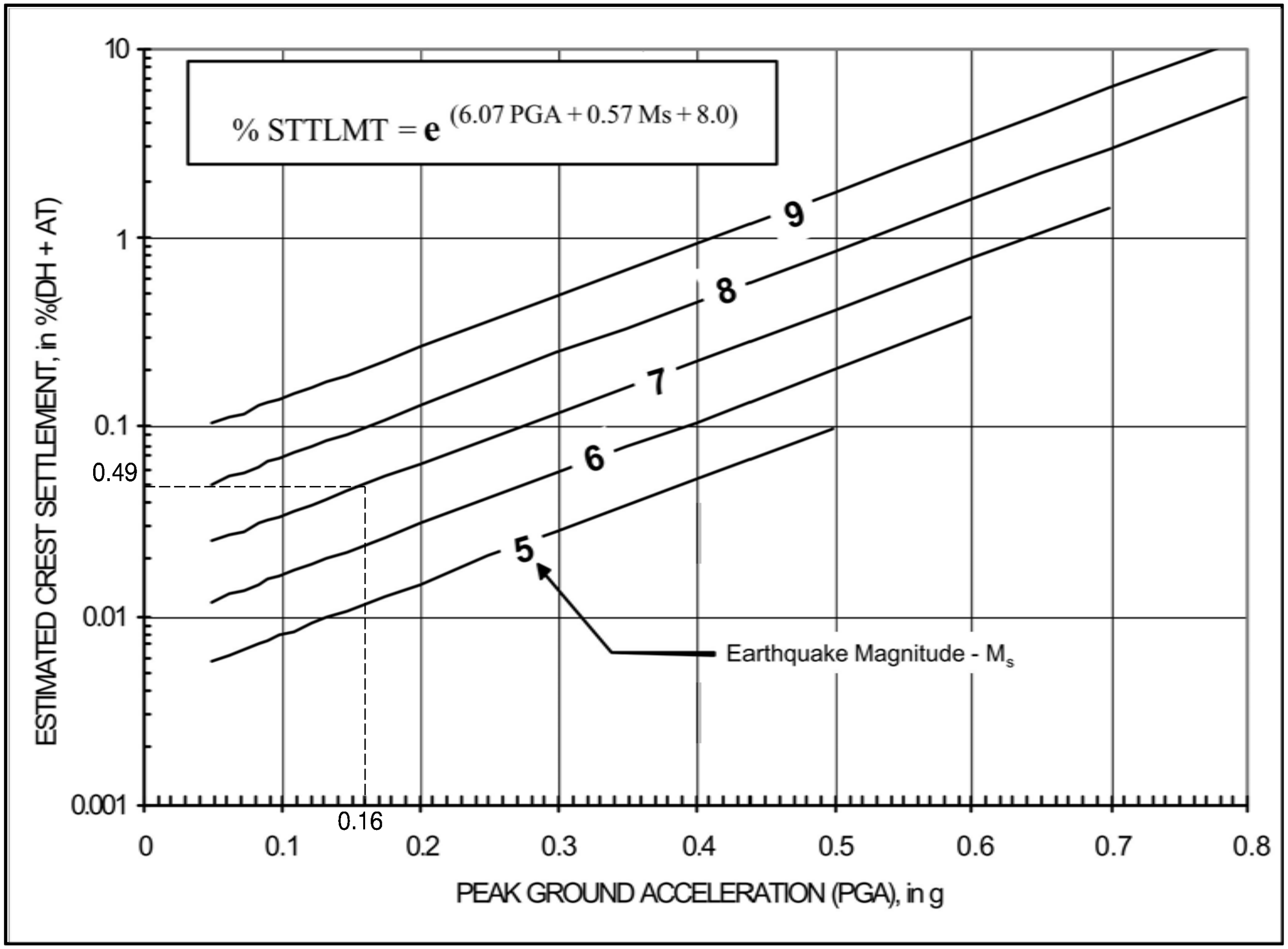

Figure 2. Chart for estimating crest settlement

REFERENCE INFORMATION:

1. J.R. SWAISGOOD, "EMBANKMENT DAM DEFORMATIONS CAUSED BY EARTHQUAKES", 2003 PACIFIC CONFERENCE ON EARTHQUAKE ENGINEERING

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SHEET TITLE: SWAISGOOD ANALYSIS - DAM DEFORMATION POTENTIAL

JOB NO. 1163.103

DRAWN BY: TDV/SCW

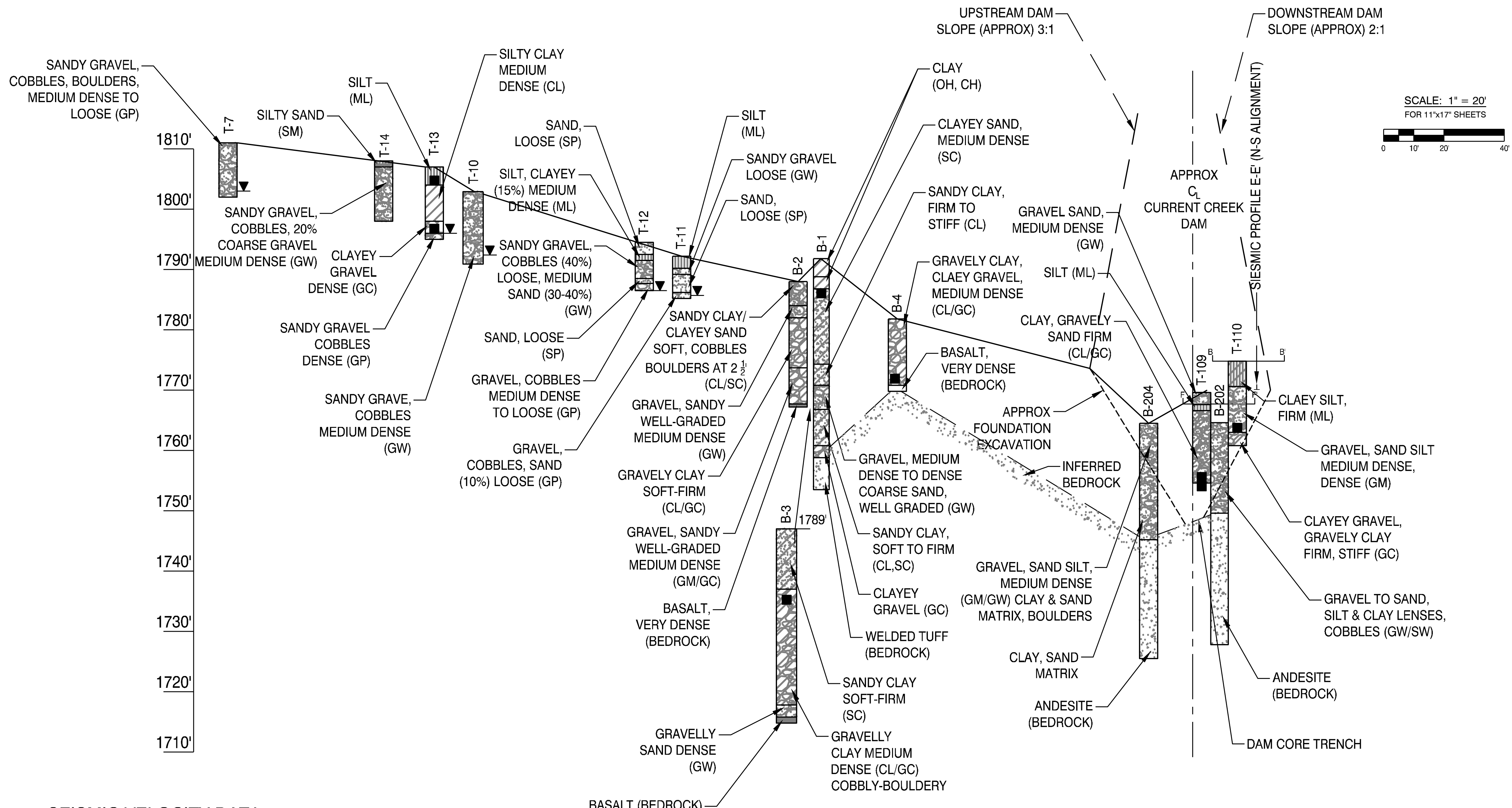
DRAWING: FIG 18B

FILE: \\freenas\NAS\HAM Engr\1163.103 WFR Dam Raise & Spillway\Engineering\Geotech Figures\200519-1163-GEO-FIGURES 1,4,6,7,9,11-15.dwg 5/27/2020 12:47 PM - Owner

CROSS-SECTION A-A'

W E

HORIZONTAL SCALE 1" = 300'



SEISMIC VELOCITY DATA

ALLUVIAL MATERIAL NEAR DAM — AVERAGE SEISMIC VELOCITY

PROFILE	LAYER	THICKNESS (FT)	FT/SEC	M/SEC
B-B'	1	19 - 22	1,400	424
B-B'	2	-	7,100	2,152
E-E'	1	17.5 - 24	1,800	545
E-E'	2	-	20,000	6,061
F-F'	1	4 - 6.5	1,700	515
F-F'	2	15 - 20	6,300	1,909
F-F'	3	-	25,000	7,576

LEGEND

N-S ⊥ E-E' SEISMIC REFRACTION PROFILE NORTH-SOUTH (APPROX)

■ LABORATORY TEST SAMPLE

▼ GROUND WATER LEVEL (APPROX)

B B' SEISMIC REFRACTION PROFILE EAST-WEST (APPROX)

F F' SEISMIC REFRACTION PROFILE EAST-WEST (APPROX)

No.	REVISION:	DATE:

DATE: 5/19/20

DRAWING STATUS: ☒ GEOTECHNICAL REPORT

H.A. McCOY ENGINEERING & SURVEYING, LLC

PROJECT: CURRENT CREEK DAM

PROJECT LOCATION: WASCO COUNTY, OR

CLIENT: YOUNG LIFE

SHEET TITLE: CROSS-SECTION OF ALLUVIAL MATERIAL IN DAM FOUNDATION

JOB NO. 1163.103

DRAWN BY: SCW

DRAWING: FIG 20

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Attachment 4-2

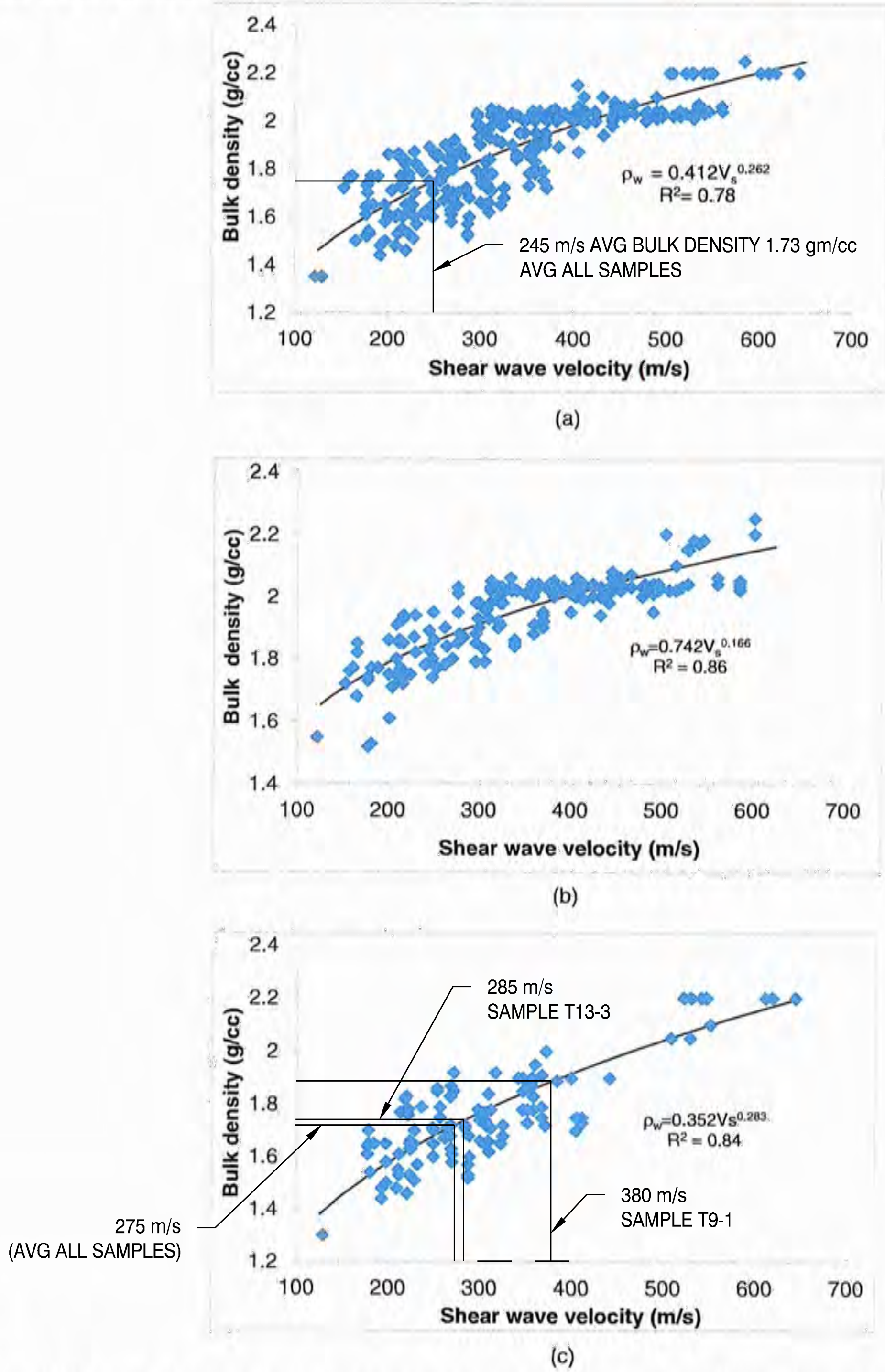

Figure 5. (a) Relation between bulk density and shear wave velocity for all soil types. (b) Relation between bulk density and shear wave velocity for fine-grained soil. (c) Relation between bulk density and shear wave velocity for coarse-grained soil.

JOB NO. 1163.103
DRAWN BY: TDV/SCW
DRAWING: FIG 21

SHEET TITLE:
CORRELATIONS BETWEEN BULK DENSITY AND SHEAR WAVE VELOCITY

PROJECT: CURRENT CREEK DAM
PROJECT LOCATION: WASCO COUNTY, OREGON
CLIENT: YOUNG LIFE

H.A. McCOY
ENGINEERING & SURVEYING, LLC
1180 SW Lake Road, Suite 201 Redmond, OR 97756
Hayes McCoy
PE, PLS, CWRE
541-923-7554 hayes@ham-engr.com

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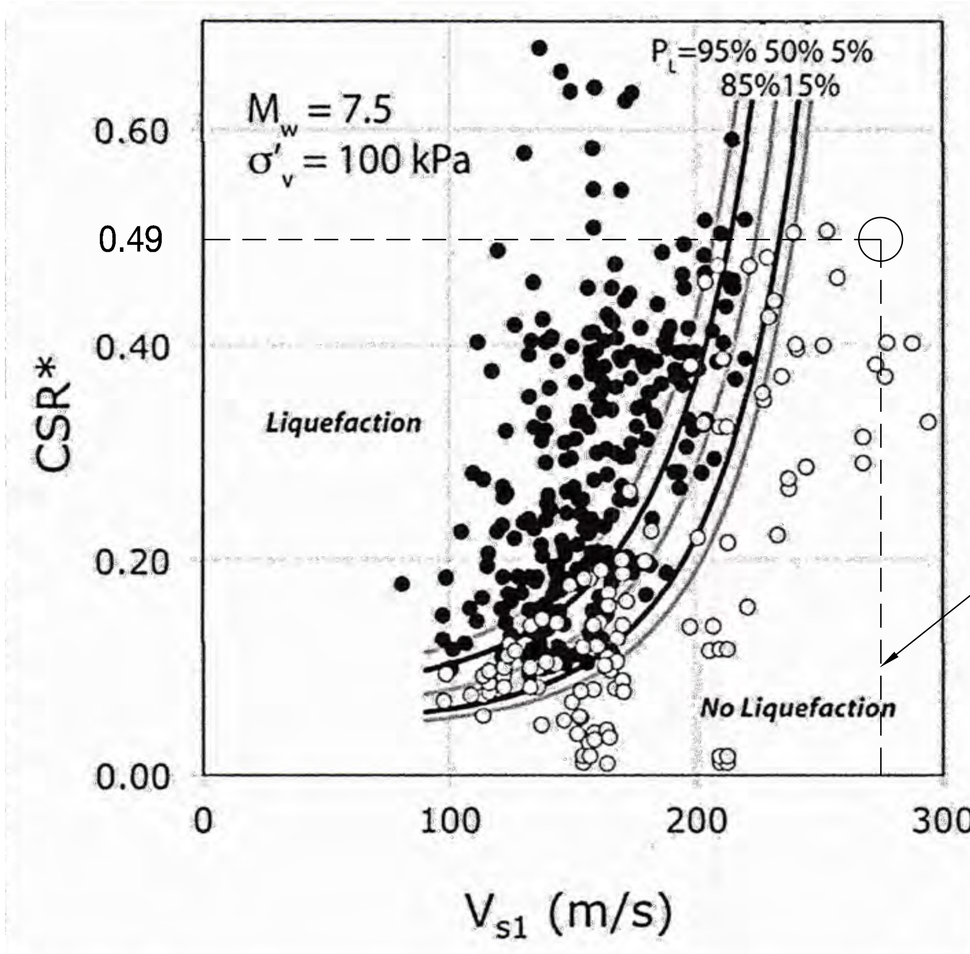
DESIGN STANDARDS NO.13
CHAPTER 13: SEISMIC ANALYSIS AND DESIGN
(U.S. BUREAU OF RECLAMATION)



275 m/s
(AVG ALL TESTED SAMPLES)

FIGURE E2.
LIQUEFACTION PROBABILITY AS A FUNCTION OF V_{S1} AND ADJUSTED CYCLIC STRESS RATIO, CSR*[8].

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DATE: 5/28/20

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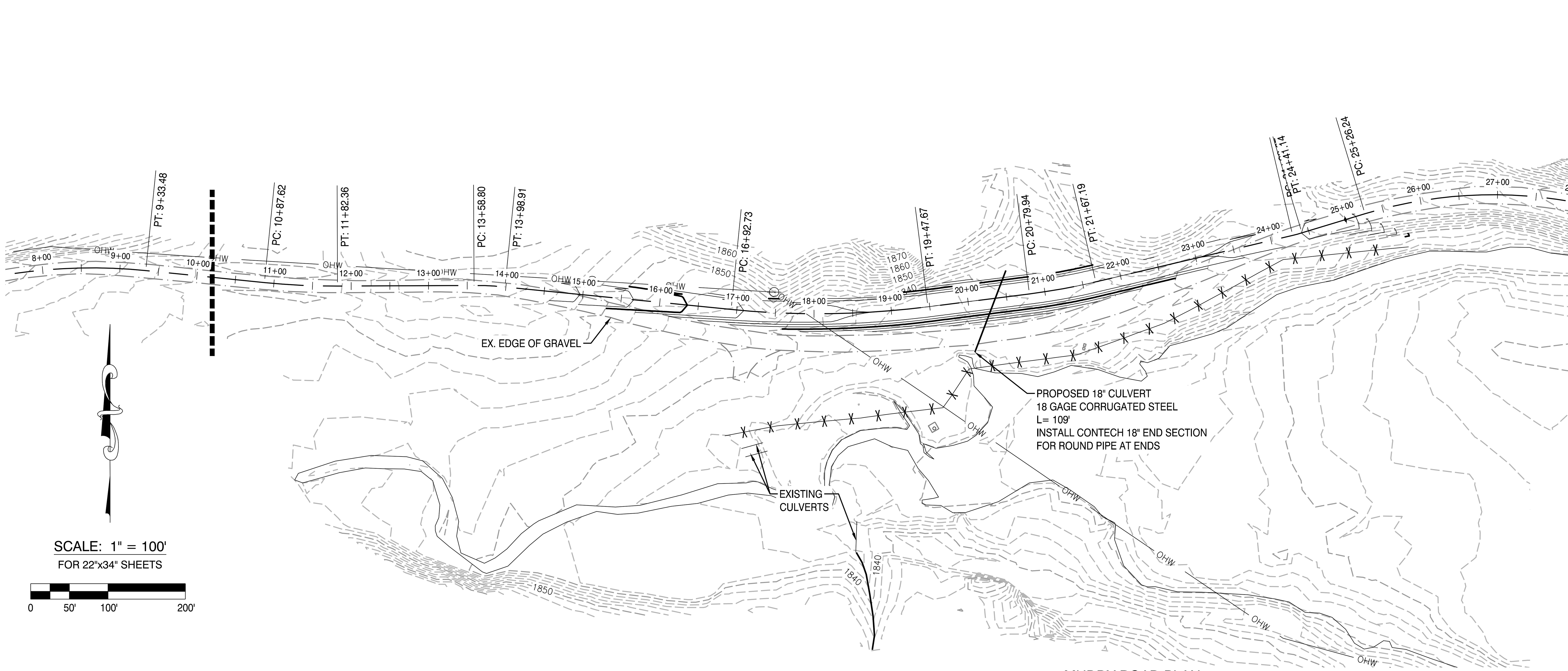
ENGINEERING & SURVEYING, LLC
1180 SW Lake Road, Suite 201 Redmond, OR 97756
Hayes McCoy
PE, PLS, CWRE
541-923-7554
hayes@ham-engr.com

PROJECT: CURRENT CREEK DAM
PROJECT LOCATION: WASCO COUNTY, OREGON
CLIENT: YOUNG LIFE

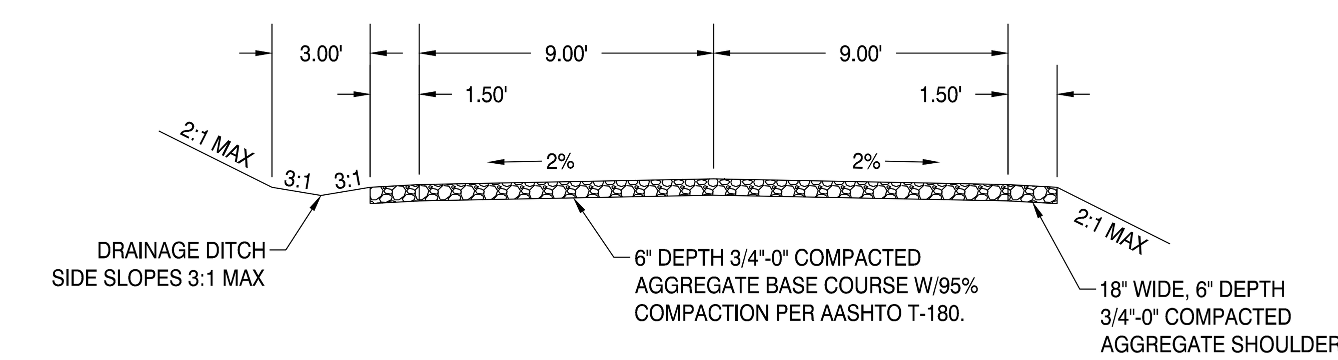
SHEET TITLE: CORRELATIONS BETWEEN CSR AND SHEAR WAVE VELOCITY

JOB NO. 1163.103
DRAWN BY: TDV/SCW
DRAWING: FIG 22

FILE: \\freenos\NAS\HAM Engr\1163.103 WFR Dam Raise & Spillway\Engineering\Geotech Figures\200519-1163-GEO-FIGURES 1,4,6,7,9,11-15.dwg 5/27/2020 12:47 PM - Owner

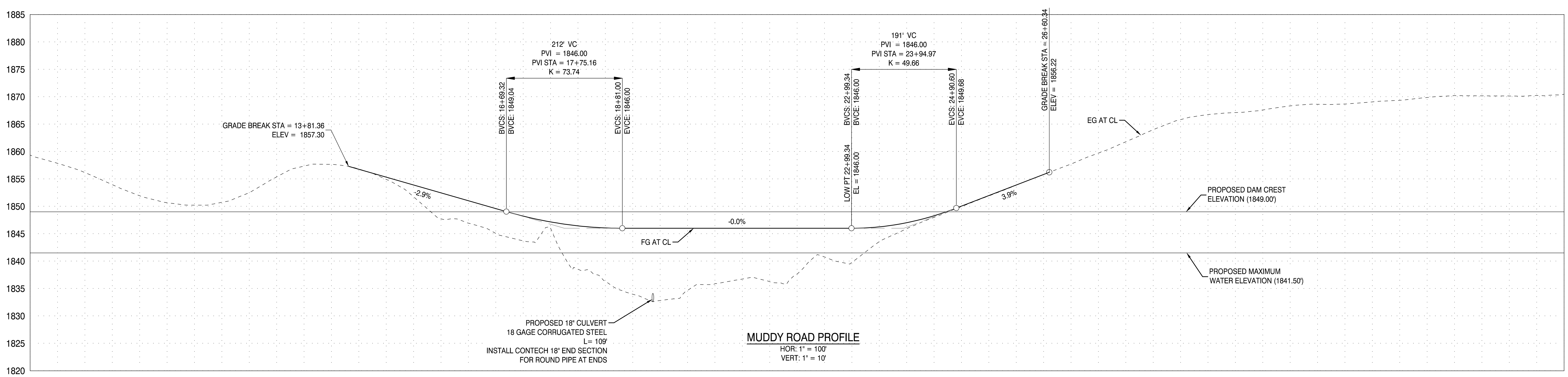


MUDDY ROAD PLAN



ROAD CONSTRUCTION SECTION

NTS



MUDDY ROAD PROFILE

HOR: 1" = 100'
VERT: 1" = 10'

PRELIMINARY
REGISTERED PROFESSIONAL ENGINEER
DAVID J. NEWTON
RENEWAL DATE: 12/31/20

DRAWING STATUS:	DATE:	No.	REVISION:	DATE:
☒ PRELIM PLANS	05/21/20			

H.A. McCOY
ENGINEERING & SURVEYING, LLC
Hayes McCoy
PE, PLS, CWRE
541-923-7554
hayes@ham-engr.com

PROJECT:
CURRANT CREEK DAM

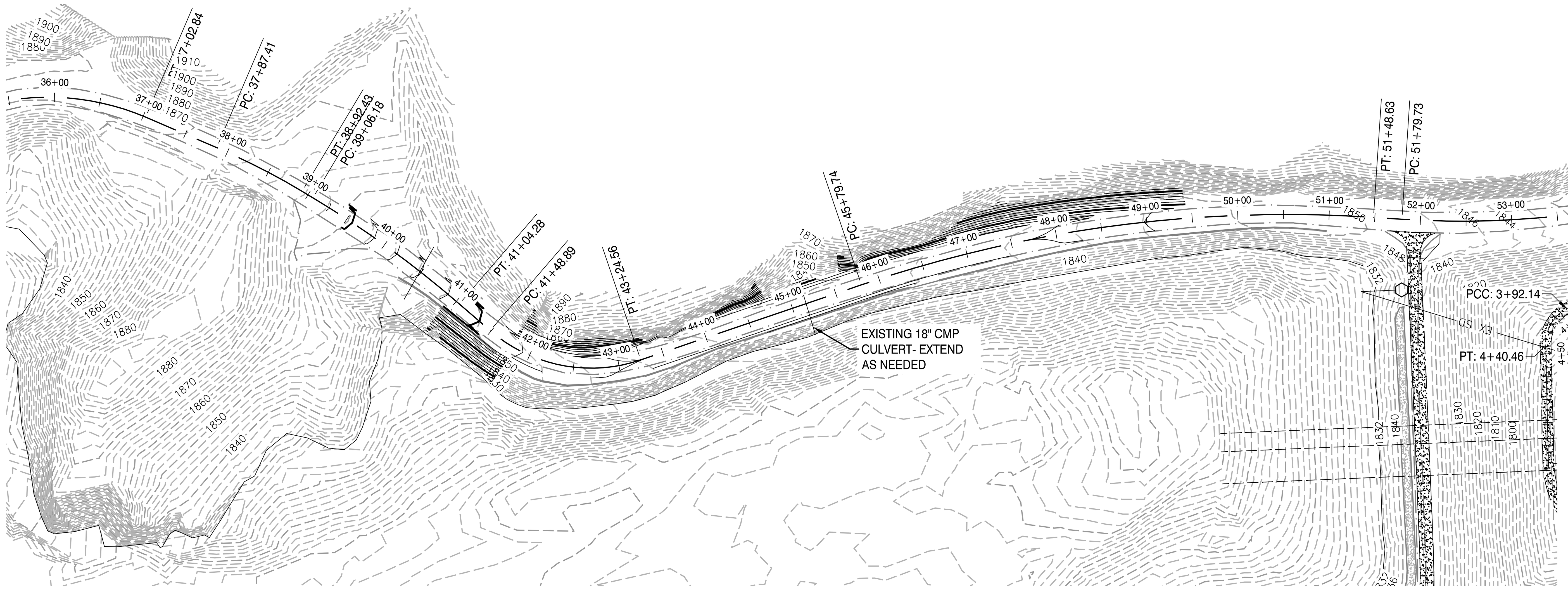
PROJECT LOCATION:
WASCO COUNTY, OR

CLIENT:
WASHINGTON FAMILY RANCH

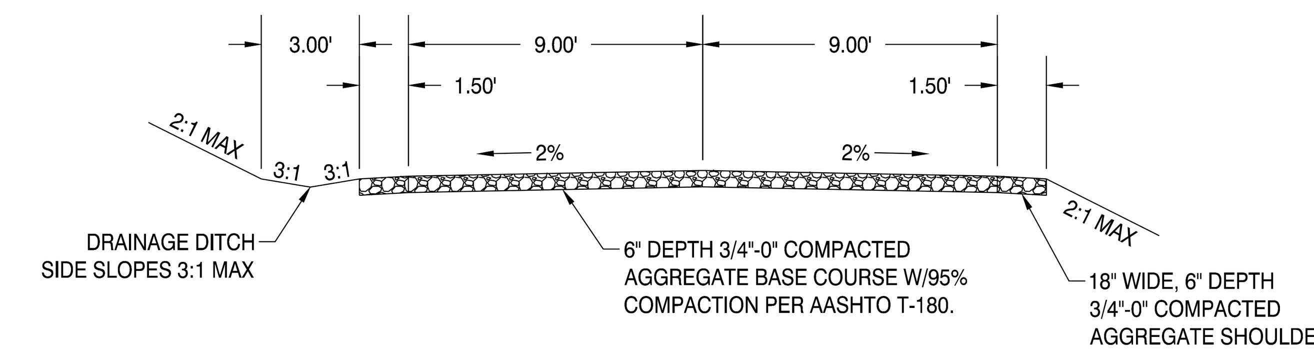
SHEET TITLE:
MUDDY ROAD PLAN
AND PROFILE STA.
8+00 TO STA. 36+00

JOB NO. 19-225
DRAWN BY: SCW
FIGURE:
FIG 23A

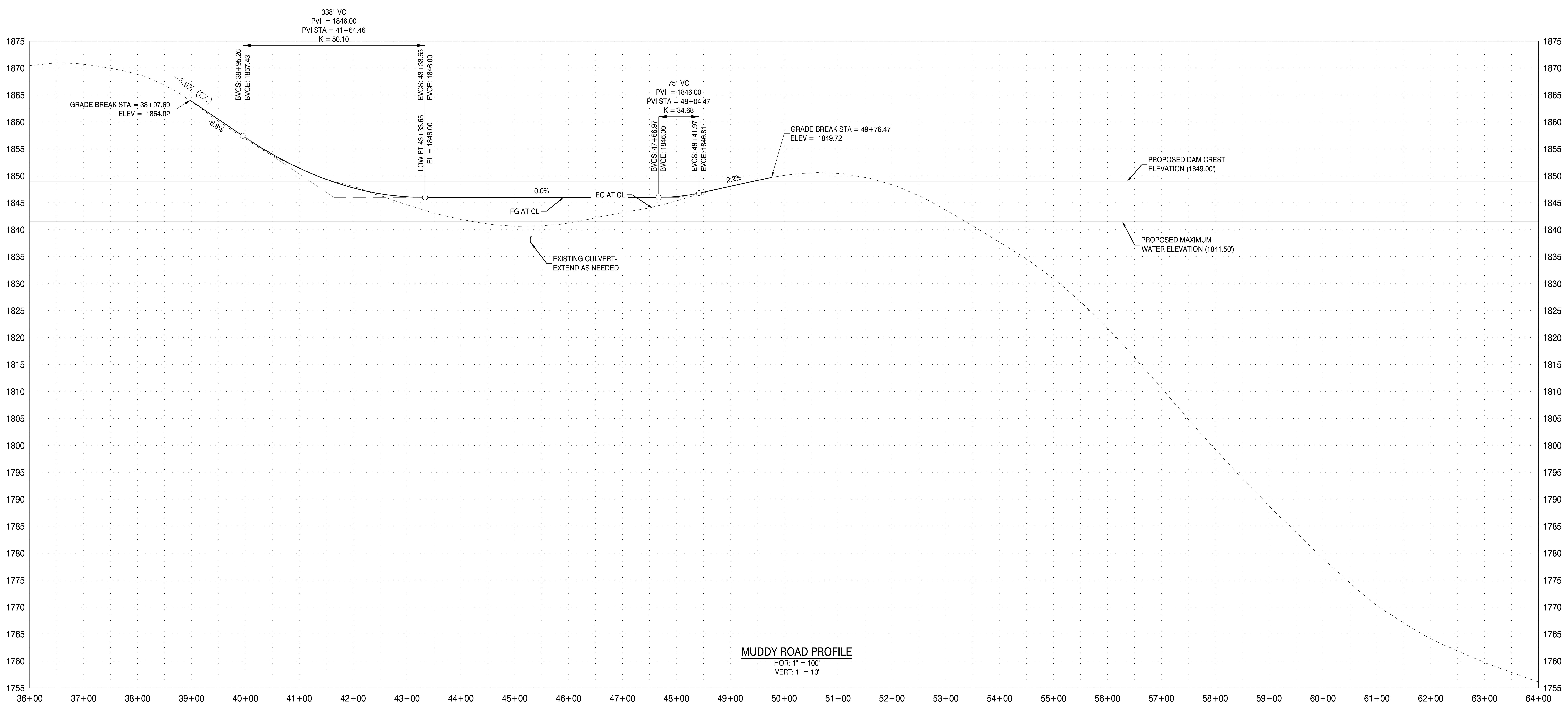
FILE: \\freenas\NAS\HAM Engr\1163.103 WFR Dam Raise & Spillway\Engineering\200522-19225-ENGR.dwg 5/27/2020 6:11 PM - Owner



MUDDY ROAD PLAN



ACCESS ROAD SECTION
NTS



MUDDY ROAD PROFILE
HOR: 1" = 100'
VERT: 1" = 10'

PRELIMINARY
REGISTERED PROFESSIONAL ENGINEER
DAVID J. NEWTON
RENEWAL DATE: 12/31/20

No.	REVISION:	DATE:

DRAWING STATUS: PRELIM PLANS DATE: 05/21/20

H.A. McCOY ENGINEERING & SURVEYING, LLC
Haydes McCoy
PE, PLS, CWRE
541-923-7554
hayes@ham-engr.com

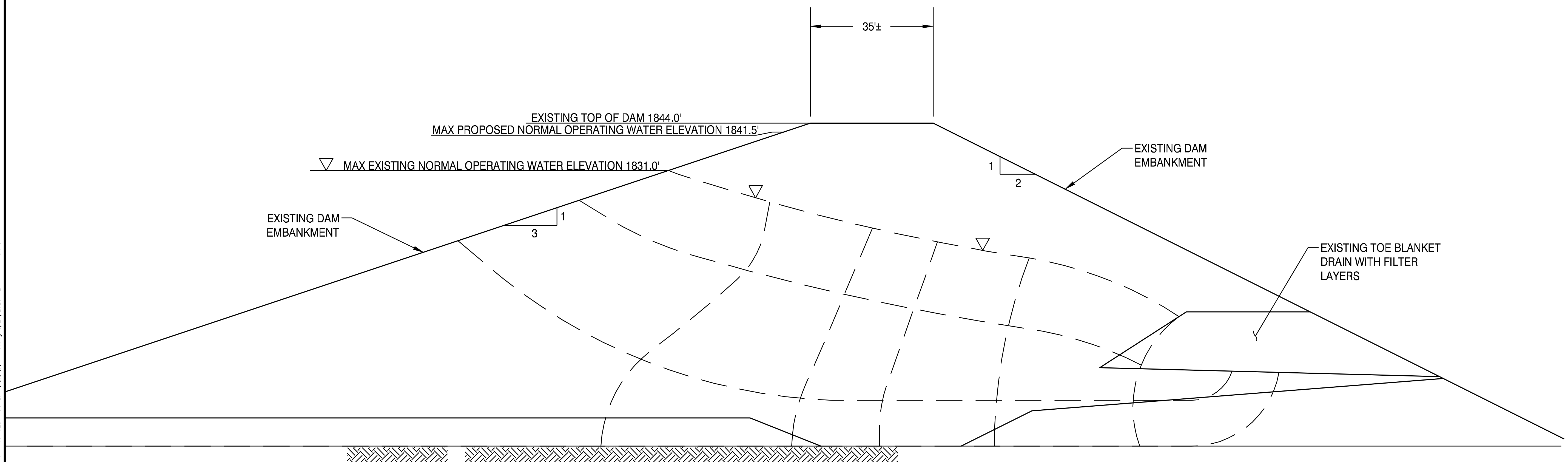
PROJECT: CURRANT CREEK DAM
PROJECT LOCATION: WASCO COUNTY, OR
CLIENT: WASHINGTON FAMILY RANCH

SHEET TITLE: MUDDY ROAD PLAN AND PROFILE STA. 36+00 TO STA. 64+00

JOB NO. 19-225
DRAWN BY: SCW
FIGURE: FIG 23B

FILE: \\freenas\NAS\HAM Engr\1163.103 WFR Dam Raise & Spillway\Engineering\200522-19225-ENGR.dwg 5/27/2020 6:11 PM -- Owner

SCALE: 1" = 30'
FOR 11"x17" SHEETS
0 15' 30' 60'



No. FLOW TUBES $N_f = 2.38$
No. HEAD DROPS $N_d = 5$

$$\frac{N_f}{N_d} = \frac{2.38}{5} = 0.48$$

DATE:
No. REVISION:
DATE: 5/19/20
DRAWING STATUS: GEOTECHNICAL REPORT

H.A. McCOY ENGINEERING & SURVEYING, LLC

PROJECT: CURRENT CREEK DAM
PROJECT LOCATION: WASCO COUNTY, OR
CLIENT: YOUNG LIFE

SHEET TITLE: FLOW NET EXISTING RESERVOIR HEAD CONDITIONS

JOB NO. 1163.103
DRAWN BY: SCW
DRAWING: FIG 24

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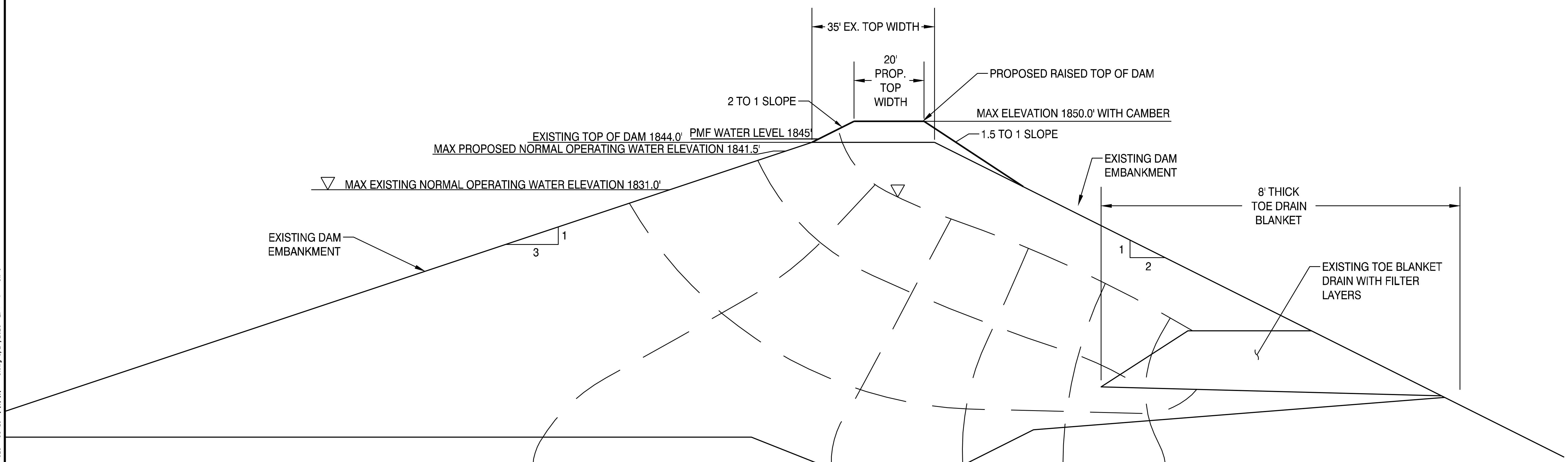

SCALE: 1" = 30'
FOR 11"x17" SHEETS
0 15' 30' 60'

No. FLOW TUBES $N_f = 2.53$
No. HEAD DROPS $N_d = 5$

$$\frac{N_f}{N_d} = \frac{2.53}{5} = 0.51$$

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☒ GEOTECHNICAL REPORT	5/19/20			

H.A. McCOY ENGINEERING & SURVEYING, LLC

PROJECT: CURRENT CREEK DAM

PROJECT LOCATION: WASCO COUNTY, OR

CLIENT: YOUNG LIFE

SHEET TITLE: FLOW NET PROPOSED RESERVOIR HEAD CONDITIONS

JOB NO. 1163.103

DRAWN BY: SCW

DRAWING: FIG 25

FILE: \\freenas\NAS\HAM Engr\1163.103 WFR Dam Raise & Spillway\Engineering\Geotech Figures\200519-1163-GEO-FIGURES 1,4,6,7,9,11-15.dwg 5/27/2020 12:47 PM - Owner

HYDROLOGY AND INFLOW DESIGN STORAGE CAPACITY AND BREACH ANALYSIS REPORT

CURRANT CREEK RESERVOIR
WASHINGTON FAMILY RANCH
WASCO & JEFFERSON COUNTIES, OREGON
Township 8S Range 18E Section 35 & Township 9S Range 18E Section 2
NOVEMBER 19th, 2019

Prepared for:

Washington Family Ranch
PO Box 220
1 Muddy Rd
Antelope, OR 97001

Prepared by

1180 SW Lake Rd. Ste. 202
Redmond, OR 97756
www.RSIengr.com

INTRODUCTION

The site evaluation report is intended to support the expansion of the Currant Creek Reservoir (Reservoir) on the border of Jefferson and Wasco Counties, Oregon. The Reservoir was originally constructed in 1983-4 and operates under Certificate 89219 (1020 ac-ft) and will be expanded under application R-88276. The existing dam has a spillway elevation of 1830 ft and a dam crest elevation of 1845 ft (NAVD88 Geoid12b). The work will include the raising of the existing dam crest to 1849 ft. and the spillway to 1840.8 ft. with a final storage capacity of 1416 acre-ft. This document and the associated analysis were performed to satisfy the requirements of OAR 690-020-0120 (Dam Breach Inundation Analysis). This report was prepared in order for Oregon Water Resource Department (OWRD) to assist in the determination of a hazard rating of the proposed dam according to OAR 690-020-0100.

PROJECT LOCATION

The project is located 11 miles S-SE of Antelope, OR on the Washington Family Ranch at a latitude of 44.8235 and a longitude of -120.5283. Figure 1 shows the location of the project in relationship to the state of Oregon.

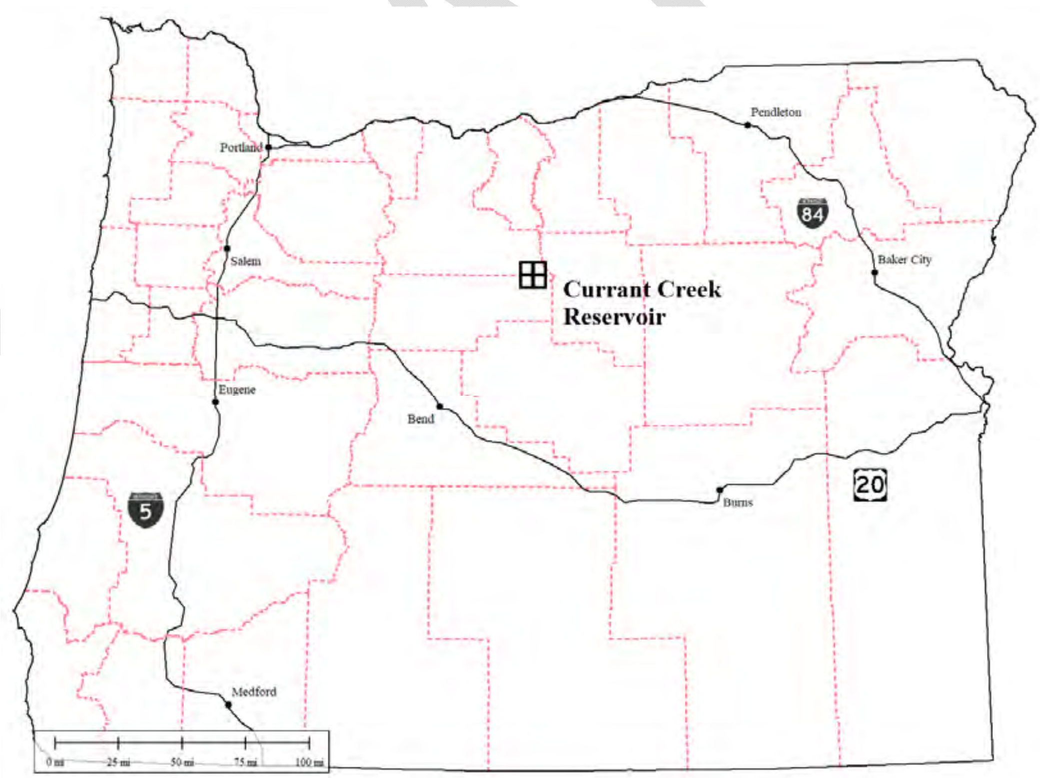

Figure 1 - Location map of Currant Creek Reservoir

SITE CHARACTERISTICS

Using the StreamStats website (streamstats.usgs.gov/ss/), the drainage area for the reservoir was delineated and computed to be 35.6 square miles with a maximum elevation of 4690 feet (Figure 2). The full StreamStats report is provided in Appendix A.

Currant Creek exits the Reservoir and flows 2.7 miles to the east where it joins with Muddy Creek. Muddy Creek then proceeds another 3.9 miles to the northeast at which point it joins with the John Day River at RM 116.3. The river corridor through this section has been heavily developed with a large number of man-made structures (buildings, airstrip, and swimming pool) in proximity to the stream system.

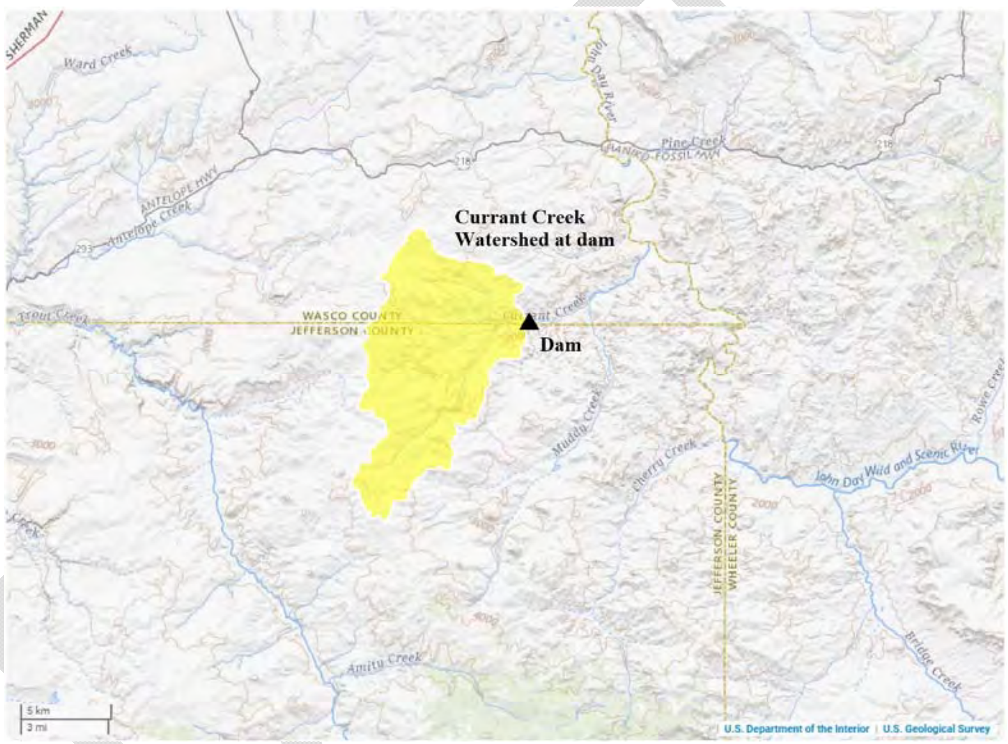

Figure 2 - Watershed of Currant Creek Reservoir from Streamstats

INFLOW DESIGN FLOOD

A Probable Maximum Flood (PMF) Analysis for the Reservoir was prepared by WEST Consultants, Inc. in 2017. The PMF event determined from this study was 11,890 cubic feet per second inflow into the Reservoir. This report is provided as an Annex at the end of this document.

STAGE AND STORAGE CAPACITY CURVE

A terrestrial and bathymetric survey of the Reservoir and surrounding area was performed in the spring of 2019 by HA McCoy Engineering and Surveying and Resource Specialists Inc. (RSI), both of Redmond, OR. Terrestrial survey was conducted with TopCon RTK GNSS/GPS. Survey points were taken through traditional rod techniques around the perimeter of the reservoir and surrounding areas. The base station raw data was recorded and processed through OPUS for final coordinate values. The bathymetric survey was conducted with a drone boat using a HydroLite-TM single beam echosounder mounted directly below a Trimble R8s GNSS receiver linked via RTK to an OPUS corrected base station.

The locations of the bathymetric survey points are shown in Figure 3.

Figure 3 - Bathymetric survey locations of Currant Creek Reservoir

Data from the echosounder was merged with the terrestrial survey points in order to generate a combined bathymetric/topographic surface. The resulting combined surface was analyzed at various water surface elevations in order to determine a stage-volume relationship and create a hypsographic curve (Figure 4).

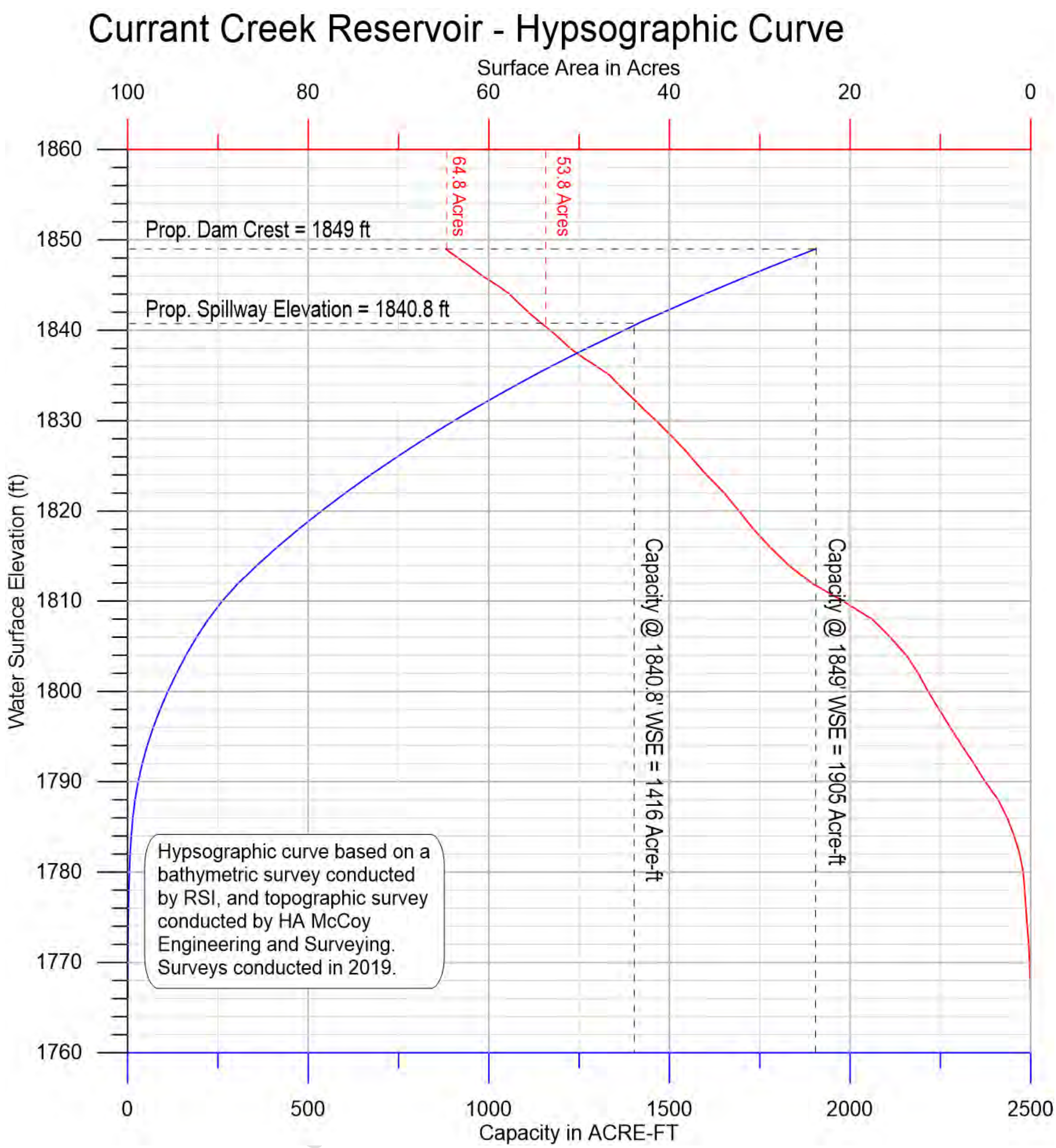

Figure 4 - Hypsographic curve of Currant Creek Reservoir

MAN-MADE STRUCTURES DOWNSTREAM OF THE RESERVOIR

An inventory of structures located downstream of the dam was conducted as part of this project. The area around Currant and Muddy Creeks downstream of the dam have been heavily developed and with many man-made structures including housing and work areas, and a large number of recreational sites. An inventory of the structures within proximity to the flood zone was done using a combination of recent UAV imagery and publicly available satellite imagery. Washington Family Ranch staff provided details on structure type/purpose/name along with frequency of use/occupation. Structures such as non-walled shade areas, pump houses, and small storage sheds were omitted from the inventory. The structure inventory is provided in Appendix B. Inundation extents at 0-ft and 2-ft are plotted for the breach analysis detailed in the later sections of this report.

EMERGENCY SPILLWAY DESIGN

The spillway on the reconstructed dam is designed to pass the PMF flow of 11,890 cfs design while maintaining 5 feet of freeboard between the water surface elevation at the PMF flow and the top of the dam crest. The area around the spillway is topographically constrained, resulting in a substantial water surface elevation rise in order to convey nearly 12k CFS. The chosen design uses a sawtooth or "labyrinth" design in order to increase the water conveyance for a fixed sill length. The sill elevation will be constructed to an elevation of 1840.8 ft, with a PMF flow over the sill resulting in a water surface elevation of 1843.8 ft. A final dam crest elevation of 1849 ft. will provide over 5 feet of freeboard between the water surface elevation during a PMF event and the top of the dam.

BREACH ANALYSIS

Two modes of failure were considered to determine the results of a catastrophic breach of the Reservoir; an overtopping failure during a PMF event and a sunny day piping failure when the reservoir is at max pool. Piping potential is a function of soil types in the dam, which will be determined in future geotechnical evaluations for design of the raised dam.

PEAK FLOW AND DURATION CALCULATIONS FOR OVERTOPPING

Failure of the dam from overtopping would likely occur from plugging of the emergency spillway to the point that the majority of inflow would be forced over the top of the dam. The peak flow and duration calculations were performed using the Washington State Department of Ecology Water Resources Program / Dam Safety Office's "Technical Note 1" developed in 2007 as a reference. Dam failure discharge flows and times were calculated using Froehlich (2008) as well as equations for erosion resistant and cohesionless soils from Fread (1981) in order to produce 3 different failure models.

DAM BREACH USING FROEHLICH (2008)

The Froehlich (2008) equations below were used to calculate the peak discharge given the anticipated parameters at the Reservoir. The PMF inflow of 11,890 CFS developed by WEST Consultants, Inc. was used as in inflow for the breach calculations.

The proposed dam crest elevation is 1849 ft, four feet higher than the current dam configuration. Based on the hypsographic curve and assuming a complete plugging of the emergency spillway, the pool storage at the height of the dam crest is 1905 acre-ft. The resulting peak flow (Q_p) is **99,579 cfs**. Using the Froehlich (2008) equations the breach time formation (T_f) is **0.331 hours (19.9 minutes)**. Based on a stored volume of 1905 acre-feet, an inflow rate of 11,890 cfs, and the peak flow of 99,579 cfs, the resulting total time to return to base flow through the reservoir is approximately **0.608 hours (36.5 minutes)**.

$$Q_P = 40.1 \times V_w^{0.295} \times H_w^{1.24}$$

$$T_f = 3.664 \sqrt{\frac{V_w}{gH_b^2}}$$

BREACH FORMATION TIME USING MACDONAL AND LANGRIDGE-MONOPOLIS

MacDonald and Langridge-Monopolis (1984) was used to calculate the the *Breach Formation Factor* (BFF) which is **212,755 ac-ft²**·based on a reservoir volume of 1905 ac-ft, inflow of 11,890 cfs and H_w = 85 ft.

$$BFF = V_w \times H_w$$

Next the volume of material eroded was calculated using the formula for erosion resistant embankment materials ($V_{m\,(er)}$) and for cohesionless embankment materials ($V_{m\,(co)}$) with the results being $V_{m\,(er)}$ = **31,653 yds³** and $V_{m\,(co)}$ = **47,479 yds³**.

$$V_{m(er)} = 2.50 \times BFF^{0.77}$$

$$V_{m(co)} = 3.75 \times BFF^{0.77}$$

A resulting breach formation time of t_{er} = **1.50 hours (90 minutes)** was determined by the breach formation time equation for dams composed of erosion resistant materials and a breach formation time of t_{co} = **1.25 hours (75 minutes)** for cohesionless materials.

$$t_{er} = 0.036 \times V_{m(er)}^{0.36}$$

$$t_{co} = 0.020 \times V_{m(co)}^{0.36}$$

DAM BREACH USING FREAD (1981)

Fread (1981) was used to estimate the dam failure peak discharge given the anticipated parameters at the Reservoir. The resulting peak flow for erosion resistant embankment materials ($Q_{p\,(er)}$) was calculated to be **51,056 cfs** and the peak flow for cohesionless materials ($Q_{p\,(co)}$) was calculated to be **79,426 cfs**.

$$Q_p = 3.1 \times W \times H_w^{1.5} \times \left[\frac{A}{A + t\sqrt{H_w}}\right]^3$$

Peak discharges and timing of the three overtopping scenarios are provided in Table 1. Macdonal and Langridge-Monopolis is abbreviated as MLM with erosion resistant abbreviated as ER and cohesionless abbreviated as CO.

Table 1 - Overtopping dam failure discharge and timings

Overtopping Failure

Model	Peak Discharge (cfs)	Time of Peak (min)	Total Time (min)
Froehlich	99579	20	36
MLM-ER	51056	90	
MLM-CO	79426	58	

PEAK FLOW AND DURATION CALCULATIONS FOR PIPING

The same equation sets can be used to examine a sunny day piping failure where the water level is set at the spillway elevation of 1840.8 feet.

The sunny day piping scenario peak discharge values are 25%-35% less than those seen during an overtopping failure (Table 2).

Table 2 - Piping dam failure discharge and timings

Piping Failure

Model	Peak Discharge (cfs)	Time of Peak (min)	Total Time (min)
Froehlich	74226	17	28
MLM-ER	33053	75	
MLM-CO	51497	48	

DAM BREACH MODEL SUMMARY

The breach models exhibited peak flows ranging from 51.1K cfs to 99.6K cfs for an overtopping failure and peak flows of 33.1K cfs to 74.2K cfs for a piping failure. In both failure scenarios, the Froehlich equations produced the highest peak discharge values. In the interest of safety and approaching the analysis in a highly conservative manner, the Froehlich numbers were used for both the overtopping and sunny-day piping failure simulations.

The Total Time of the breach was not calculated for Macdonal and Langridge-Monopolis models as a standard triangular model would violate continuity (i.e. greater discharge than total volume) based on the calculated peak times. This is due to the MLM model using an envelope equation for breach time prediction which will tend to overestimate the actual time (USACOE, 2014). If these models were to be used for the HEC-RAS simulations, some adjustments would need to be made with either a power function during the formation time or modifying the timing to fit a standard triangular flow model. As the chosen models for both scenarios are from Froehlich this analysis is unnecessary.

HYDRAULIC MODELING

Hydraulic modeling was performed for the breach analysis from the dam at the Reservoir to the confluence with the John Day River. The modeling effort was stopped at the confluence with the John Day River according to the direction of Keith Mills with OWRD (pers. comm. 5/8/19).

LiDAR data was available for the lower 1.5 miles of Muddy Creek, whereas the upstream portion consisting of Muddy and Currant Creeks was surveyed with photogrammetric techniques using a UAV. The UAV was equipped with a survey grade L1/L2 GNSS receiver and photo positions were computed using PPK resulting in centimeter level accuracy for photo positions. Ground control points were used throughout the survey areas with assessed vertical accuracies less than 2* GSD (ground sampling distance) which translated to absolute accuracies around three inches. Surfaces were produced from the drone imagery using Agisoft Photoscan Professional. In the overlapping area between the LiDAR and the UAV surfaces, vertical agreement was good, with typical discrepancies of 0.1- 0.3 ft (Figure 5). The DEM of the modeled area is shown in Figure 6.

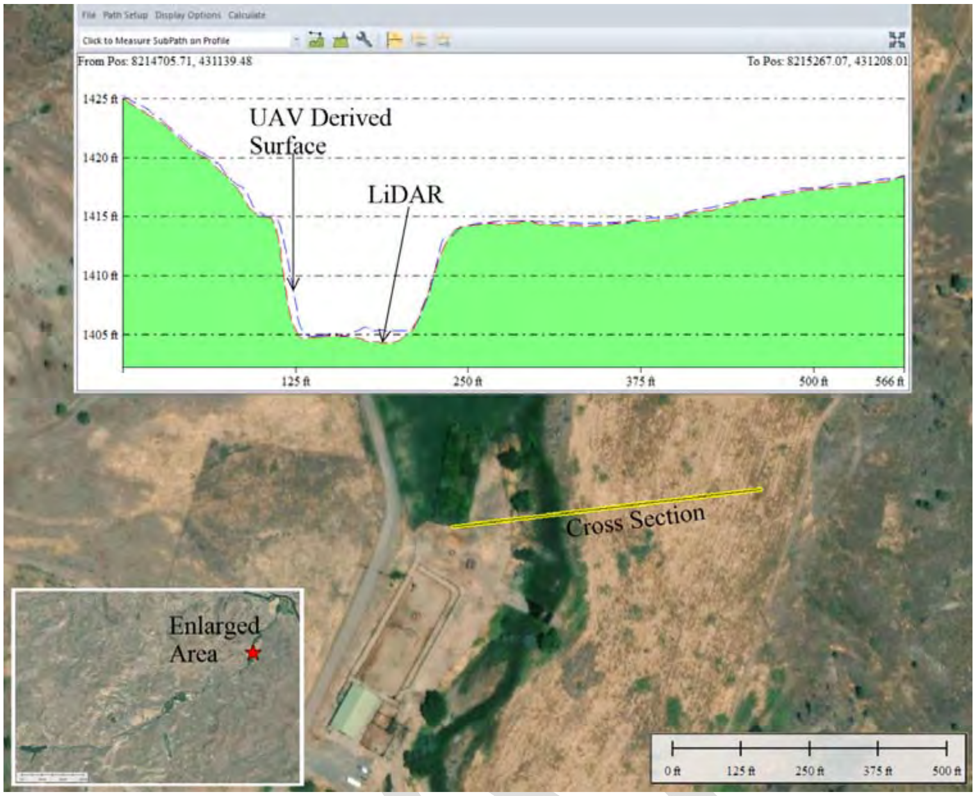

Figure 5 - Surface comparison of UAV and LiDAR

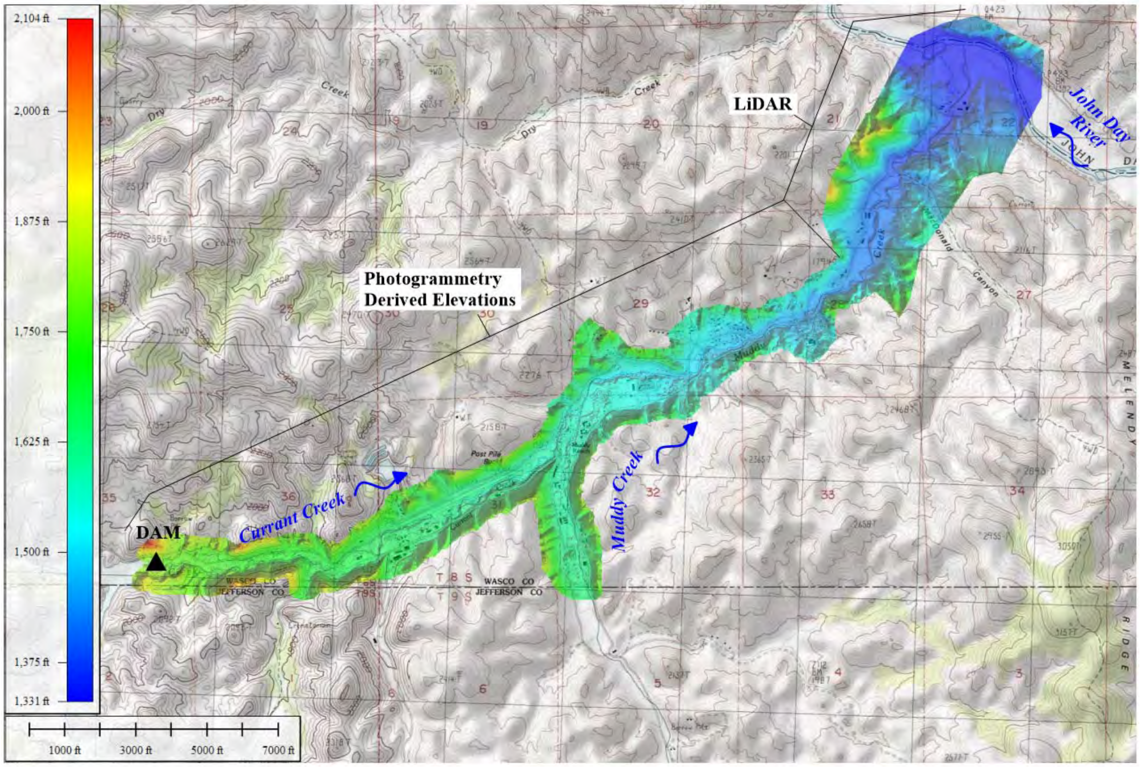

Figure 6 – DEM of area modeled for breach analysis

Modeling Approach

The latest version of HEC-RAS (5.0.7) was used to construct the hydraulic models of the dam breach. The area from the dam to the confluence of the John Day River was modeled as a 2D flow area with a hydrograph boundary condition corresponding to the dam breach hydrograph located at the dam face (Figure 7 and Figure 8) and a normal depth boundary condition at the confluence.

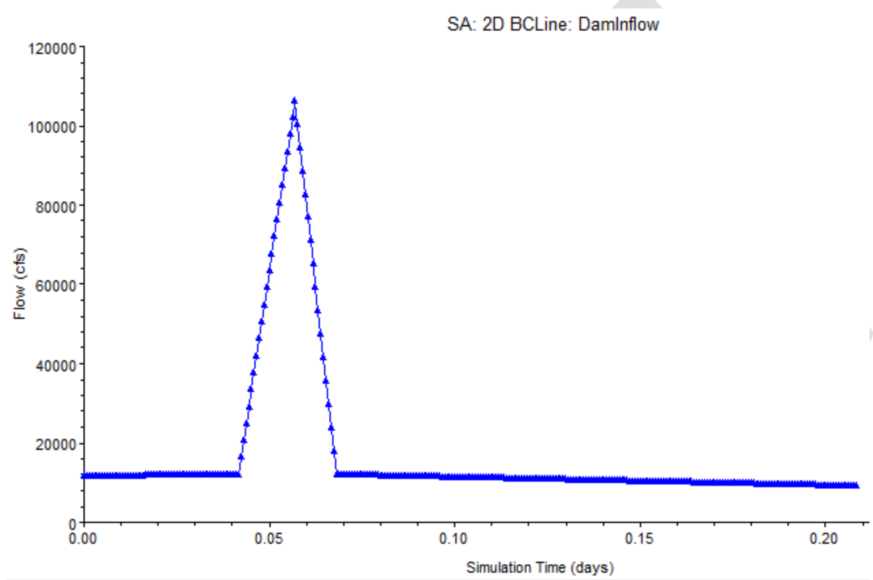

Figure 7 - Hydrograph of inflow boundary condition representing the overtopping dam breach flow

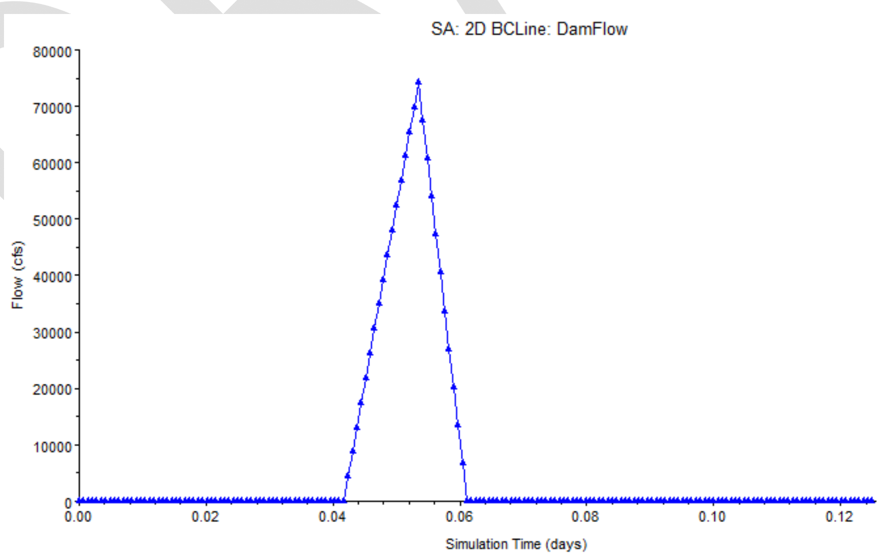

Figure 8 - Hydrograph of inflow boundary condition representing the sunny-day piping breach flow

The HEC-RAS mesh was generated at 20-foot grid spacing resulting in ~124,000 computation cells. Initial conditions of a PMF flow from the dam (11,890 cfs) were allowed to develop for an initial condition period of one hour. Additional inflows from Muddy Creek and other smaller drainages were disregarded for the model. The simulation was run using the full momentum equation set, a computation interval of half a second, and a total model run time of 3 hours that allowed for the peak flow to fully exit the modeled domain. The area downstream of the dam is generally lacking of heavy vegetation and what near-stream vegetation is present would likely be washed away during the event due to the expected depths and velocities. The model was run at three Manning's roughness coefficients (0.03, 0.035, and 0.04) for the overtopping scenario in order to analyze the sensitivity of this parameter.

A total of 10 road crossings are present over Currant and Muddy Creeks between the dam and the confluence with the John Day River (Figure 9). All of these crossings are single, double, or triple large diameter (5'+) culverts except for crossing number 8, which is a bridge. When looking at the scale of the breach event, the proportion of flow that the culverts could convey is very small; additionally, the likelihood of plugging is high due to the amount of debris expected in the flow. Consequently, the surface model of the culvert areas included the road surface with no conveyance accounted for through the culverts.

The open channel under the bridge is relatively confined with a width of approximately 32' and an approximate depth of 8' from the stream channel to the underside of the bridge. Running a rough flow calculation based on a channel slope of 1.3% and a roughness coefficient of 0.035, the channel is capable of conveying approximately 4,000 cfs underneath the bridge. Due to the likelihood of debris becoming lodged on/under the bridge and the relatively small proportion of flow that could be conveyed underneath the structure, the modeling was performed in the most conservative manner with the road surface in place.

The automated point classification algorithms used to clean the UAV point cloud data were very good at removing vegetation from the bare earth model, however many of the roofs of the larger buildings were identified as ground points and subsequently incorporated into the surface model. Modeling the buildings in this manner effectively treats the building areas as obstructions not capable of passing flow. The alternative method would be to create regions of high Manning's n around the buildings in order to model greatly reduced velocities through the structures.

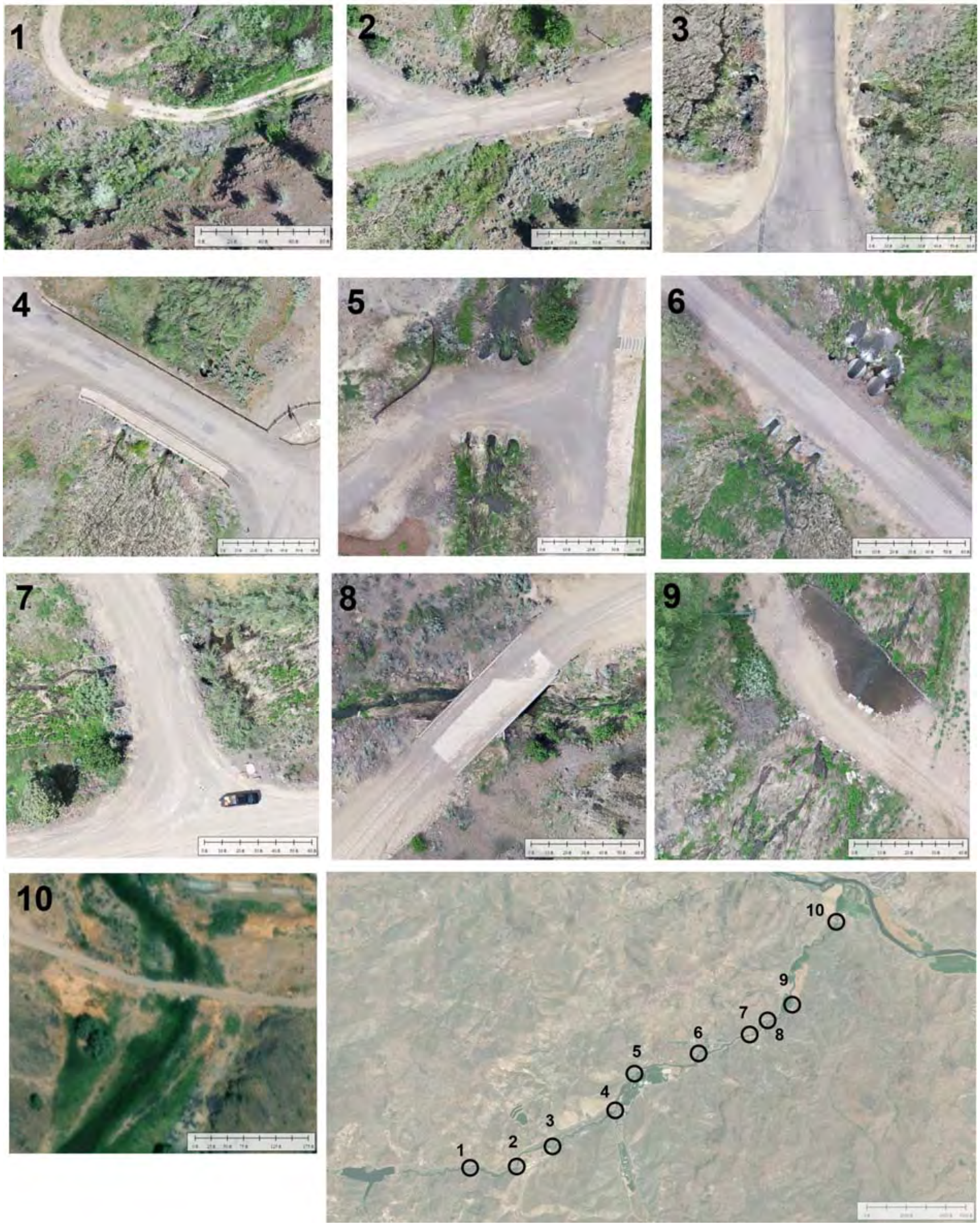

Figure 9 - Road crossings over Currant and Muddy Creeks within the modeled area

Modeling Results

Maximum depth and velocity were analyzed for three Manning's n values (0.03, 0.035, & 0.04) for the overtopping scenario and a piping scenario conducted with a Manning's n value of 0.04.

Full-scale maximum depth and velocity results for the overtopping scenario with a Manning's n of 0.04 are shown in Figure 10 and Figure 11. A detailed examination of the maximum depths and velocities is provided in Appendix C for the overtopping scenario and Appendix D for the piping scenario.

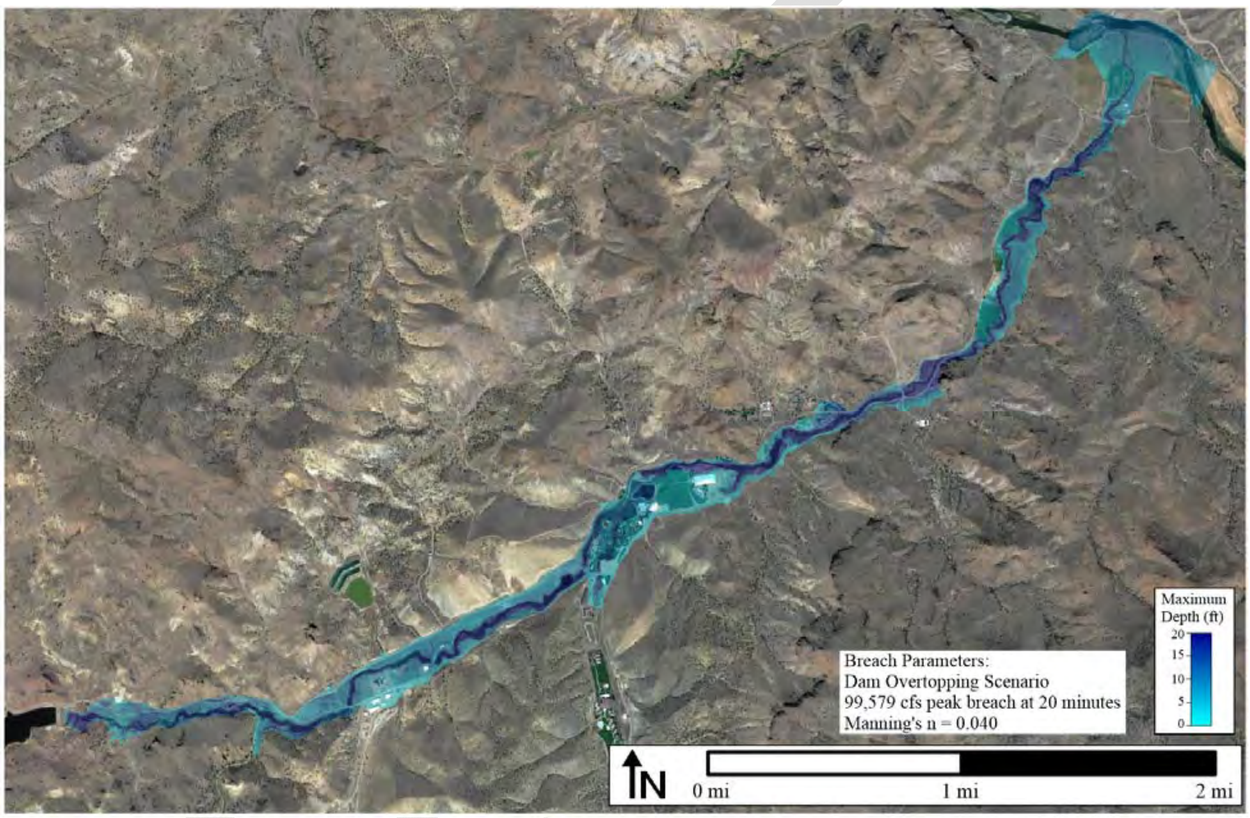

Figure 10 - Depth and velocity plots of HEC-RAS model output from the overtopping scenario

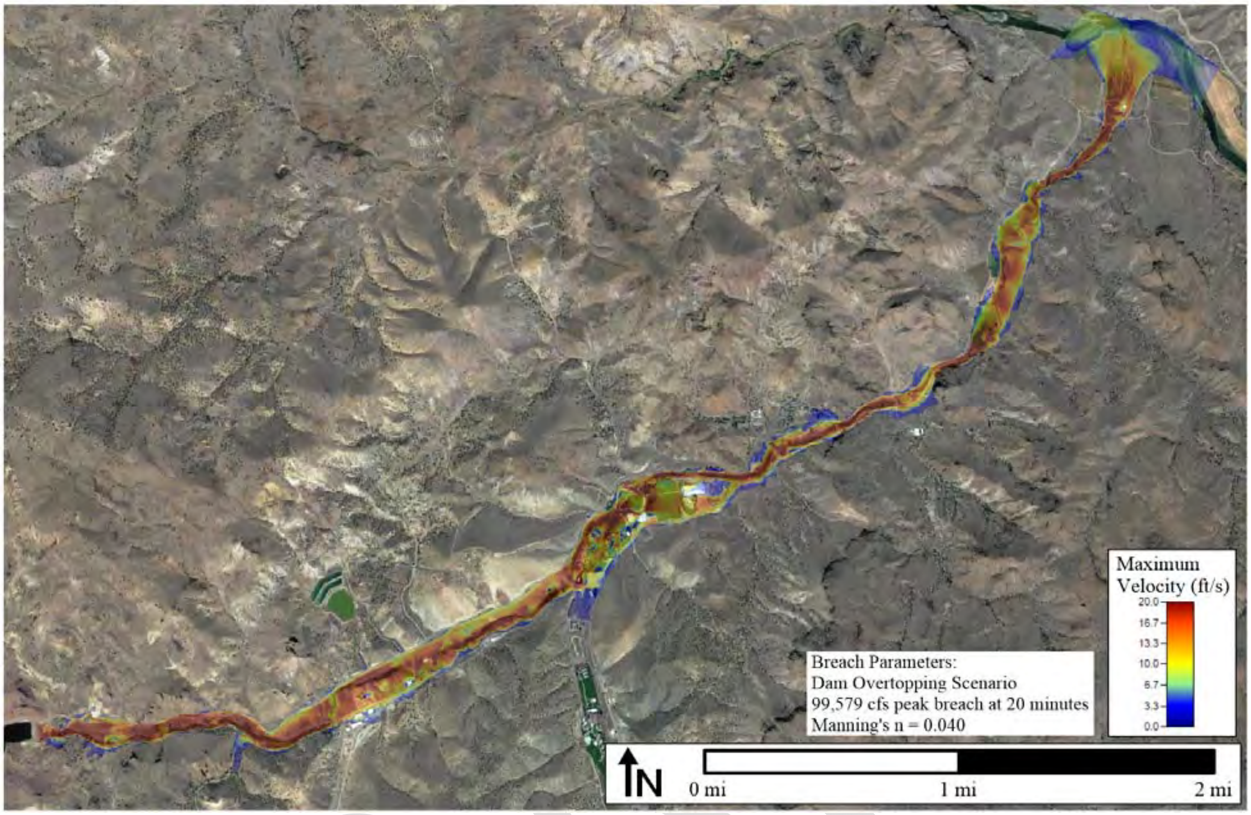

Figure 11 - Maximum velocity during overtopping breach scenario, Manning's n = 0.04

The modeling results show a large number of frequently occupied structures will be inundated to a depth greater than 2-ft during both dam failure scenarios. The two areas that will see the largest impact to man-made structures will be the area near the confluence of Currant and Muddy Creeks and the employee housing area located approximately a mile downstream of the confluence of the two creeks. The modeled maximum water depth for these two areas is shown in Figure 12.

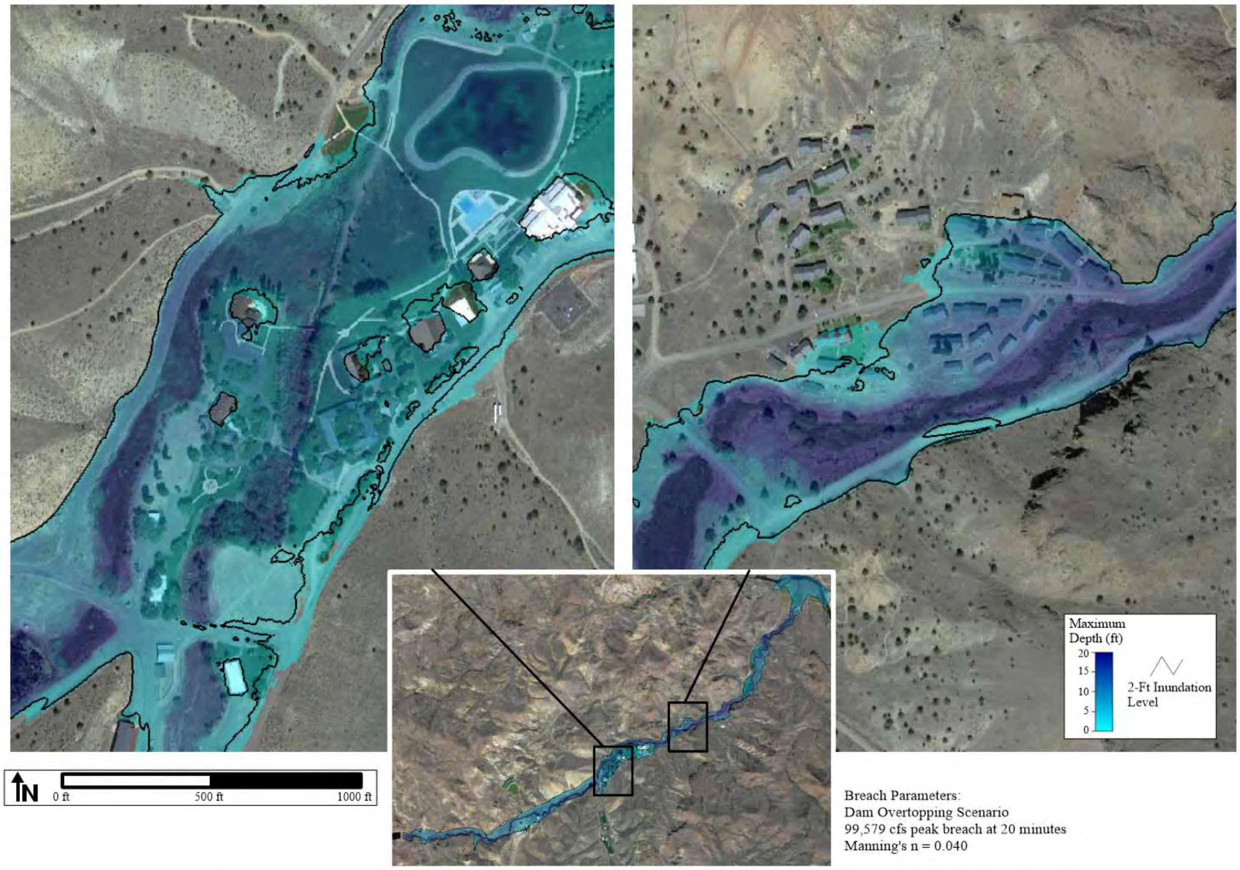

Figure 12 - Modeled maximum water depth (overtopping scenario) for two heavily impacted areas within Washington Family Ranch

Manning's n Sensitivity Analysis

The overtopping model was run with Manning's n values of 0.03, 0.035, and 0.04 in order to analyze the sensitivity of the model to this parameter. As expected, as the roughness is increased the water depth and inundation extent are also increased. Two cross-sections (one taken along Currant Creek, one taken along Muddy Creek) with varying roughness coefficients are shown in Figure 13. A difference grid was created by subtracting the maximum water surface elevation of the simulation run with a Manning's n of 0.030 from the maximum water surface elevation run with a Manning's n of 0.040. The resultant grid is shown in Figure 14 along with a profile from near the dam to the confluence with the John Day River. The water surface elevation difference between the three runs generally varied less than a foot with an average difference of about 0.8 ft.

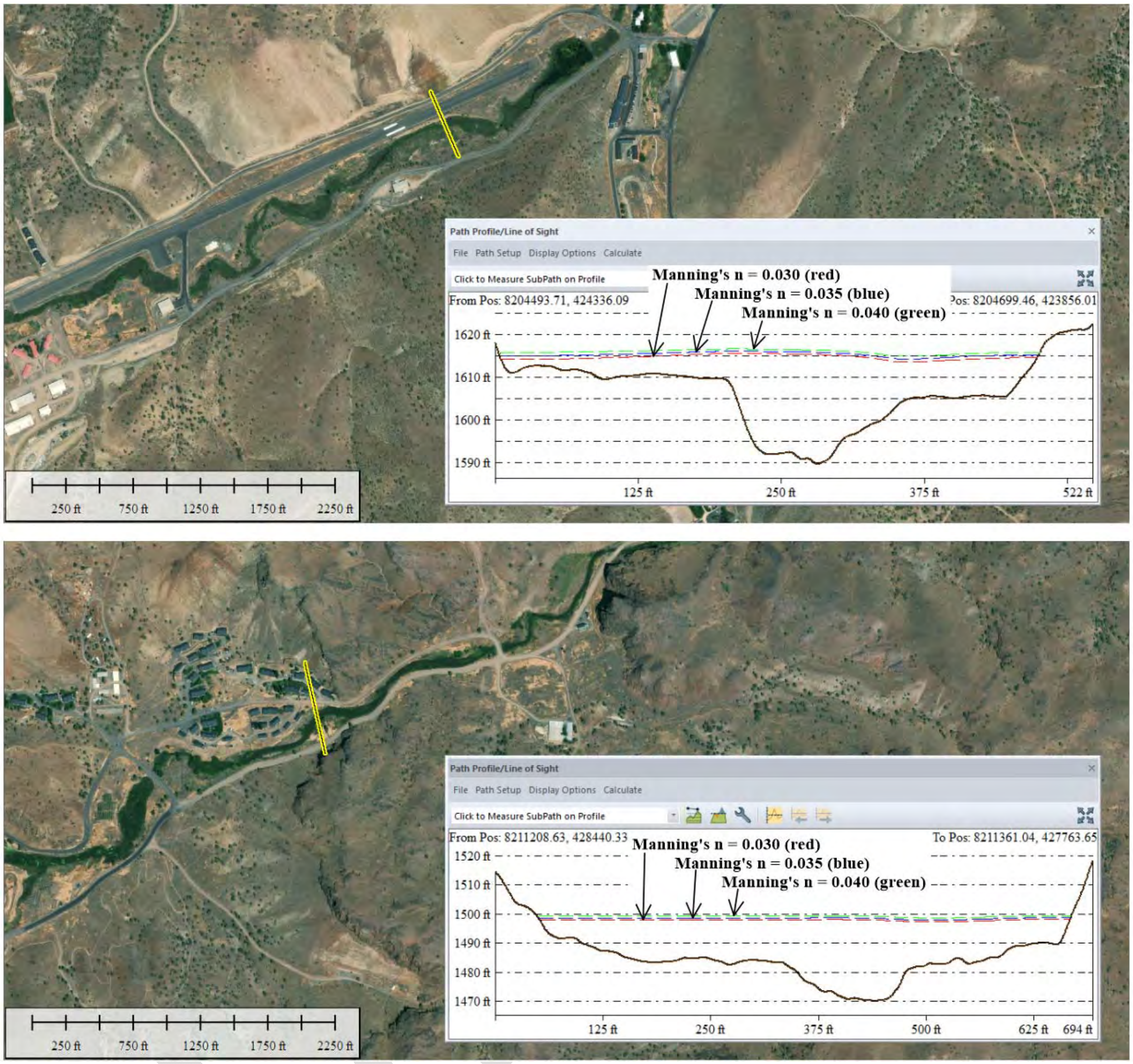

Figure 13 - Two cross-sections showing the maximum water surface elevation for three Manning's n values

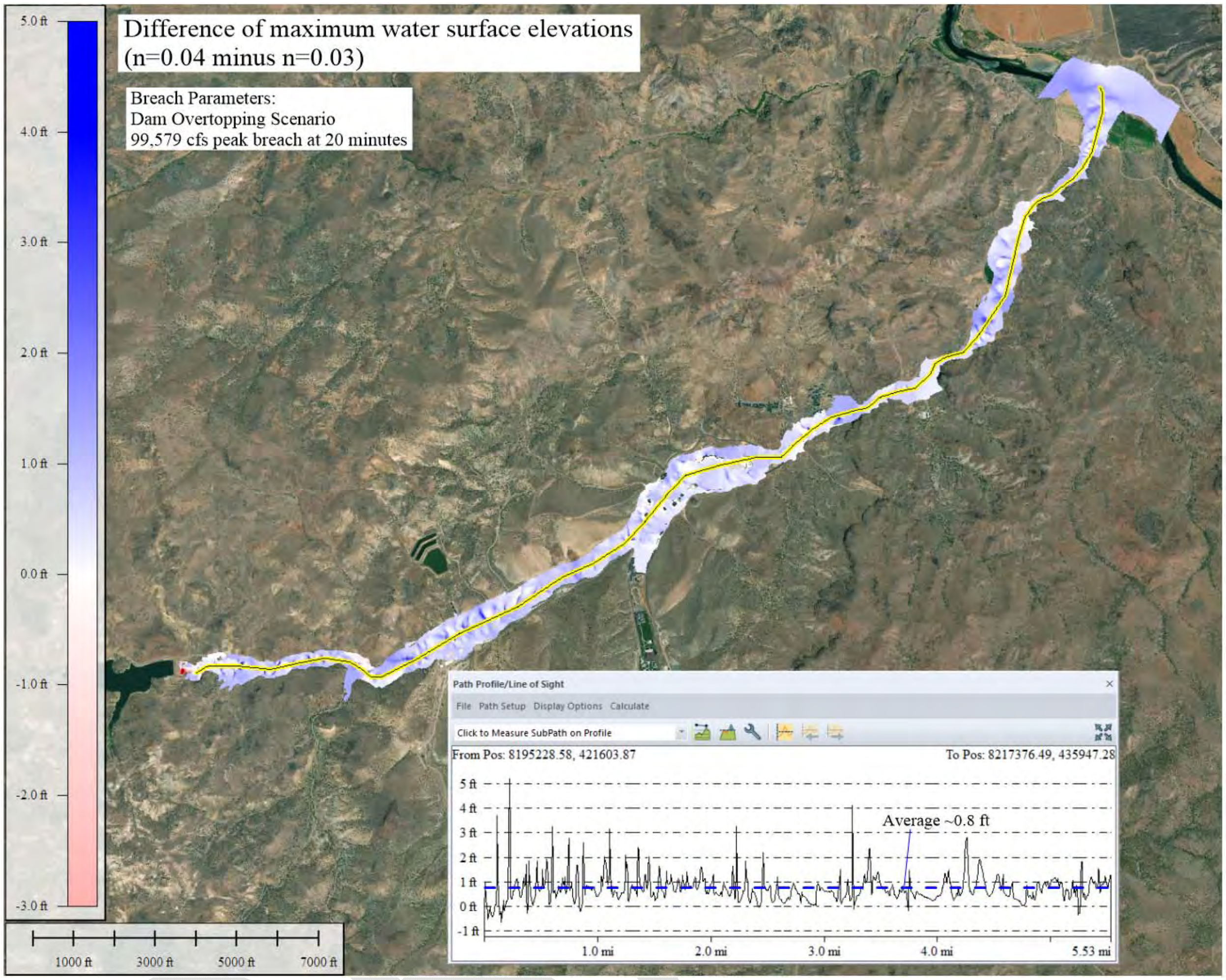

Figure 14 - Water surface elevation difference for Manning's n of 0.04 and 0.03, plan and profile

SUMMARY OF FINDINGS

This document reports the stage and storage capacity curve and breach analysis of a proposed dam expansion project on Currant Creek Reservoir in Jefferson and Wasco Counties.

Three models of overtopping failure were explored combined with a PMF event of 11,890 cfs. A HEC-RAS model was constructed of the most conservative (highest peak flow) breach scenario with a peak event discharge of 99,579 cfs. A sensitivity analysis of the Manning's n coefficient was conducted for the overtopping scenario showing water surface elevation differences of approximately 0.8 ft.

Three breach models were calculated for a sunny-day piping failure of the dam. The most conservative model produced a peak flow of 74,226 cfs. This breach scenario was modeled in HEC-RAS using a Manning's n value of 0.04.

All models demonstrated that a large number of frequently occupied man-made structures would be inundated to depths greater than 2 feet, with some structures seeing inundation depths in excess of 10 feet.

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