City of West Linn   Planning & Development • 22500 Salamo Rd #1000 • West Linn, Oregon 97068

DEVELOPMENT REVIEW APPLICATION

For Office Use Only		
STAFF CONTACT John Boyd	PROJECT NO(S). CU-15-02/ OR 15-06 / DR 15-07	
NON-REFUNDABLE FEE(S) ⊖	REFUNDABLE DEPOSIT(S) ⊖	TOTAL ⊖

Type of Review (Please check all that apply):

- ☐ Annexation (ANX)
- ☐ Appeal and Review (AP) *
- ☒ Conditional Use (CUP)
- ☒ Design Review (DR) CLASS II
- ☐ Easement Vacation
- ☐ Extraterritorial Ext. of Utilities
- ☐ Final Plat or Plan (FP)
- ☐ Flood Management Area
- ☐ Hillside Protection & Erosion Control
- ☐ Historic Review
- ☐ Legislative Plan or Change
- ☐ Lot Line Adjustment (LLA) */**
- ☐ Minor Partition (MIP) (Preliminary Plat or Plan)
- ☐ Non-Conforming Lots, Uses & Structures
- ☐ Planned Unit Development (PUD)
- ☐ Pre-Application Conference (PA) */**
- ☐ Street Vacation
- ☐ Subdivision (SUB)
- ☐ Temporary Uses *
- ☐ Time Extension *
- ☐ Variance (VAR)
- ☐ Water Resource Area Protection/Single Lot (WAP)
- ☐ Water Resource Area Protection/Wetland (WAP)
- ☐ Willamette & Tualatin River Greenway (WRG)
- ☐ Zone Change

Home Occupation, Pre-Application, Sidewalk Use, Sign Review Permit, and Temporary Sign Permit applications require different or additional application forms, available on the City website or at City Hall.

Site Location/Address: 6111 SKYLINE DRIVE

Assessor's Map No.: 21E25AD

Tax Lot(s): 07100

Total Land Area: 3.23 ACRES

Brief Description of Proposal: REPLACEMENT OF MUNICIPAL DRINKING WATER SUPPLY RESERVOIR

Applicant Name: (please print) LANCE CALVERT
Address: 22500 SALAMO ROAD
City State Zip: WEST LINN, OR 97068
Phone: 503-722-5500
Email: LCALVERT@WESTLINNOREGON.GOV

Owner Name (required): (please print) CITY OF WEST LINN
Address: 22500 SALAMO ROAD
City State Zip: WEST LINN, OR 97068
Phone:
Email:

Consultant Name: (please print) MURRAY, SMITH & ASSOCIATES, INC. (TOM BOLAND)
Address: 121 SW SALMON ST
City State Zip: PORTLAND, OR 97204
Phone: 503-225-9010
Email: TOM.BOLAND@MSA-EP.COM

1. All application fees are non-refundable (excluding deposit). **Any overruns to deposit will result in additional billing.**
2. The owner/applicant or their representative should be present at all public hearings.
3. A denial or approval may be reversed on appeal. No permit will be in effect until the appeal period has expired.
4. **Three (3) complete hard-copy sets (single sided) of application materials must be submitted with this application. One (1) complete set of digital application materials must also be submitted on CD in PDF format. If large sets of plans are required in application please submit only two sets.**

* No CD required / ** Only one hard-copy set needed

RECEIVED
MAY 28 2015
By

The undersigned property owner(s) hereby authorizes the filing of this application, and authorizes on site review by authorized staff. I hereby agree to comply with all code requirements applicable to my application. Acceptance of this application does not infer a complete submittal. All amendments to the Community Development Code and to other regulations adopted after the application is approved shall be enforced where applicable. Approved applications and subsequent development is not vested under the provisions in place at the time of the initial application.

Applicant's signature: [signature]   Date: 5/28/15   Owner's signature *(required)*: [signature]   Date: 5/28/2015

BOLTON RESERVOIR REPLACEMENT

Conditional Use Permit, Class I & II Design Review

Submitted by City of West Linn

BOLTON RESERVOIR REPLACEMENT

Conditional Use Permit, Class I & II Design Review

Submitted by City of West Linn

Application File Number: PA-15-05

Proposal:
Replace the existing 100-year old covered drinking water reservoir with a new partially buried prestressed concrete reservoir meeting current seismic design standards. Replace the existing pump station flat roof with a pitched roof more in character with the residential neighborhood.

APPLICANT:	City of West Linn
Owner Representative:	Lance Calvert, Public Works Director
Project Manager:	Erich Lais, Assistant City Engineer
Project Planner:	John Boyd, Planning Manager
Project Designer:	Murray, Smith & Associates, Inc.
PROJECT LOCATION:	6111 Skyline Drive
LEGAL DESCRIPTION:	Assessor's Map 21E25AD, Tax Lot 07100
SITE AREA:	140,700 square feet
ZONING:	R-10 (Single Family Residential Detached/10,000 square foot minimum lot size)
ZONING OVERLAYS:	None
COMPREHENSIVE PLAN:	Low Density Residential
PERMITS REQUESTED:	Conditional Use Permit Class I Design Review (Pump Station Roof Replacement) Class II Design Review (Reservoir Replacement)

CITY OF WEST LINN, PUBLIC WORKS DEPARTMENT
BOLTON RESERVOIR REPLACEMENT
CONDITIONAL USE AND CLASS I & II DESIGN REVIEW

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   2. A copy of the letters to officers of the associations and to property owners within 500 ft.
   3. An affidavit of mailing and a copy of the mailing list containing the names and addresses of such owners and residents;
   4. A copy of the required posted notice, along with an affidavit of posting;
   5. A copy of the minutes of the meetings, produced by the neighborhood association.
   6. A copy of neighborhood association meeting minutes comments
   7. An audiotape of the meeting
Q. Arborist Report
R. Building material samples
   1. Decorative fence product information
   2. Reservoir exterior wall shotcrete sample photos
S. Preliminary Stormwater Management Plan
T. "Geotechnical Investigation and Site-Specific Seismic Hazard Study 4-MG Bolton Reservoir West Linn, Oregon", GRI, January 23, 2015.
U. "Geotechnical Engineering Services, Bolton Reservoir Seismic Landslide Evaluation, West Linn, Oregon," Cornforth Consultants, Inc., January 22, 2015.

CITY OF WEST LINN
PUBLIC WORKS DEPARTMENT
BOLTON RESERVOIR REPLACEMENT

CONDITIONAL USE AND CLASS I AND II DESIGN REVIEW

I. INTRODUCTION

This document is provided as part of the City of West Linn's application for a Conditional Use Permit and Class II Design Review for the proposed Bolton Reservoir Replacement project. The applicant proposes replacement of the existing Bolton Reservoir, a 2.5 Million Gallon (MG) covered municipal water storage reservoir, with a new 4.0 MG partially buried concrete water storage reservoir (water storage tank). The proposed site is an existing City owned site with a municipal water storage reservoir and pump station located at 6111 Skyline Drive, on property zoned R-10 (Single-Family Residential Detached) as defined in the West Linn Community Development Code (CDC) Chapter 11. A water storage tank (reservoir) is a major utility as defined in CDC Chapter 2, Definitions, in which water storage tank is included in the list of examples for the definition of "Utility, major". The proposed reservoir, as a major utility, is an allowed conditional use in the R-10 zone as described in CDC 11.060, Conditional Uses, which identifies "9. Utilities, major" as a conditional use allowed in the zoning district. This application also includes installation of a new roof on the Bolton Pump Station to address roof maintenance issues. The proposed roof construction requires a Class I Design Review per CDC 55.020(A)(5): "Minor modifications and/or upgrades of pump stations, reservoirs, and storm detention facilities.

II. BACKGROUND

The proposed site is the existing Bolton Reservoir and Pump Station Site located on approximately 3.23 acres of property owned by the City of West Linn. The site is developed and used for drinking water system infrastructure. Existing structures include the 2.5 MG Bolton Reservoir which provides water storage for the City; the Bolton Pump Station which pumps water to customers at higher elevations; the Old Bolton Pump Station which contains piping and valving, and instrumentation and control equipment for the site; and a storage building. The structures at the site occupy approximately 29,000 square feet (0.66 acres).

The site topography includes a large relatively flat area that slopes from Skyline Drive at approximately 5 percent to a point north of the reservoir. The northerly edge of the site slopes down at a 1h:1v (horizontal: vertical) slope. The steeper sloped area is predominately covered by deciduous trees and invasive English ivy and blackberry. There is a stand of Douglas-fir trees located adjacent to Skyline Drive. The bulk of the site vegetation is grass. The site perimeter is fenced on three sides with a 6-foot tall chain link fence topped with three strands of barbed wire, and a sliding access gate of off Skyline Drive. There is no fencing along the northern side of the property. A gravel access road connects Skyline Drive with a paved area in front of the pump station. The existing reservoir also has a 6-foot tall chain link perimeter fence topped with three strands of barbed wire. The adjacent lands are developed single-family residential lots.

Given the reservoir's functional limitations, condition, and age, its replacement is recommended in the City Council adopted Water System Master Plan (WSMP) completed in 2008.

Applicant Response to Approval Criteria

III. CONDITIONAL USE NARRATIVE

The following are the Applicant's responses to the approval criteria of Chapter 60 of the city of West Linn Community Development Code (CDC). The Site Plan and map required in Chapter 60.080 is attached and should be reviewed for reference with this narrative.

60.070 APPROVAL STANDARDS AND CONDITIONS

60.070(A): The Planning Commission shall approve, approve with conditions, or deny an application for a conditional use, except for a manufactured home subdivision in which case the approval standards and conditions shall be those specified in CDC 36.030, or to enlarge or alter a conditional use based on findings of fact with respect to each of the following criteria:

60.070(A)(1): The site size and dimensions provide:
60.070(A)(1)(a): Adequate area for the needs of the proposed use;

The site is approximately 3.23 acres and currently contains the existing 2.5 MG Bolton Reservoir, existing Bolton Pump Station, Old Bolton Pump Station and existing storage building. As shown on the Site Plan in the Appendices, the site is roughly rectangular in shape, approximately 350 feet by 400 feet. The proposed reservoir structure will occupy an area of approximately 0.54 acres, which is slightly smaller than the 0.64 acre footprint of the existing reservoir. The existing reservoir, old pump station and storage building will be demolished and removed as part of the proposed reservoir project. The site is of adequate size to accommodate the replacement reservoir structure and associated facilities, including the stormwater detention pond and water quality swale, while leaving adequate space for a future replacement storage building and maintenance vehicle access to the proposed improvements and existing pump station. The location of the proposed reservoir on the site provides code compliant setbacks from property lines on all sides.

60.070(A)(1)(b): Adequate area for aesthetic design treatment to mitigate any possible adverse effect from the use on surrounding properties and uses.

The proposed reservoir will be partially buried, with approximately 5 feet of concrete wall exposure on the south side facing Skyline Drive, increasing to approximately 15 feet of concrete wall exposure on the north side. An elevation view of the reservoir wall exposure is shown on the Concrete Reservoir Section drawing (Figure 6) in the Appendices. The proposed reservoir location is approximately 56 feet from the nearest property line (to the west). The exposed concrete reservoir wall will have a topcoat of shotcrete, which has a gray sandy textured finish. When viewed from the site frontage, the top of the reservoir roof will be several feet lower than Skyline Drive. The site has adequate area for aesthetic design treatments which will include proposed vegetative screening along the back and side property lines, and native landscaping along the front of the site, as shown on the Landscape Plan (Figure 7) in the Appendices. Existing trees along the frontage will be preserved to the maximum extent possible. Proposed site screening and landscaping will include removal of invasive English ivy and blackberry, removal of unhealthy or undesirable trees as recommended by the project arborist, and restoration with drought tolerant native species. The trees proposed for planting as landscaping and screening along the side and back property lines include Incense Cedar, Douglas-fir and Western Red Cedar. Vine Maples and assorted native shrubs are proposed for planting under the existing Douglas-fir stand to the south and southeast of the reservoir, to replace the non-native grass with an appropriate understory to improve wildlife habitat.

60.070(A)(2): The characteristics of the site are suitable for the proposed use considering size, shape, location, topography, and natural features.

The proposed replacement reservoir is locationally dependent. The existing Bolton Reservoir and Pump Station site characteristics are suitable for the proposed reservoir. The site has been a reservoir site and served as the hub of the City's water system since 1915 when the existing reservoir was constructed. The proposed use is not changing from its current use. The site size, approximately 3.23 acres, and rectangular shape are suitable for the proposed replacement reservoir and associated facilities. The site generally has flat topography at the required elevation for the proposed water storage reservoir making it suitable for the proposed use.

The location of a water storage reservoir is governed primarily by the ground elevation of the area the reservoir must serve. The reservoir structure must be placed at an elevation to provide adequate water system pressure to customers and fit the water system hydraulic conditions. In general, the proximity of a reservoir site location relative to the existing water system infrastructure also has a direct correlation to the feasibility of the site to be integrated into the existing water system, and affects the magnitude of project costs depending on the need for additional water piping and pumping facilities if required.

The City's water system is supplied with treated Clackamas River water from the South Fork Water Board (SFWB) Water Treatment Plant (WTP) located in Oregon City. Finished drinking water flows from the SFWB WTP by gravity through a 30-inch diameter transmission main to the SFWB Division Street Pump Station. This pump station boosts water from the transmission main to the City of Oregon City's Mountainview Reservoir (overflow elevation of 490 feet) and to the City's Bolton Reservoir (overflow elevation of 440 feet). Supply to the City from the discharge of the pump station is transmitted through a 24-inch diameter transmission main that extends west through Oregon City and across the I-

205 bridge into the City. From the west side of the bridge, the transmission main continues to the intersection of Buse Street and Broadway Street, and then to the Bolton Reservoir as a distribution main that also provides water service directly to customers.

The City of West Linn's water system is configured around the Bolton Reservoir as the hub of the system, with this facility serving as the terminal reservoir for the SFWB supply to the City. Supply and distribution piping, booster pump stations that pump to higher elevation pressure zones and pressure reducing facilities that serve lower elevation pressure zones have all been planned, designed and constructed to function optimally with supply from gravity storage at the Bolton Reservoir site. This operational configuration allows City Operations staff maximum control of the system and provides the greatest ability for continued water service in an emergency. The key to the effective operation of this system is the ground elevation at the existing reservoir site, which allows the reservoir to be filled by the pressure provided by the Division Street Pump Station or the Mountainview Reservoir under the existing system's hydraulic constraints.

The WSMP recommends raising the overflow elevation of the new reservoir to 450 feet to provide more effective service to the Bolton pressure zone from the reservoir when the SFWB Division Street Pump Station is not in operation, and to provide improved suction pressure to the Bolton Pump Station. The existing Bolton Pump Station experiences low suction cut out at an existing water level of approximately 435 feet, limiting the volume of stored water that can be pumped to only 1.0 MG. Raising the overflow elevation to 450 feet will increase the volume of stored water that can be pumped to the Horton pressure zone, while the remaining volume will be available to supply emergency storage to the City's lower pressure zones by gravity. Constructing a replacement for the City's terminal reservoir at a much higher elevation would not be feasible due to the complexities of the required system modifications and corresponding order-of-magnitude increase in cost.

The site has the topography and ground elevation necessary to accommodate the replacement reservoir that has a proposed floor elevation of 425 feet and maximum operating water level of 450 feet needed to meet water system hydraulic requirements to serve the Bolton Pressure Zone and supply the existing Bolton Pump Station to serve the Horton Pressure Zone. The existing Bolton Pressure Zone is directly connected to the SFWB transmission main which delivers water from the City's primary water supply. The existing Bolton Reservoir is supplied by water distribution piping from the SFWB transmission main. The Bolton Pump Station is also located at the existing reservoir site, and pumps water from the Bolton Reservoir to the Horton Pressure Zone and Reservoir. The existing site was selected by the City as the most suitable site after completing a Reservoir Siting Alternatives Analysis to investigate other potential site alternatives, followed by the subsequent geotechnical investigation and site specific seismic hazard study which confirmed the geotechnical suitability of the site.

Most of the site has existing structures, or graveled and paved access ways. The bulk of the site vegetation is non-native grass. The stand of Douglas-fir trees adjacent to Skyline Drive lacks desirable understory vegetation and the northerly sloped area is predominately covered by deciduous trees and invasive English ivy and blackberry. The proposed use at the site is suitable considering the site's limited natural resources. Existing trees along the frontage will be preserved to the maximum extent possible and enhanced with appropriate native understory vegetation. Proposed site screening and landscaping will include removal of invasive species and restoration and reforestation with drought tolerant native species

60.070(A)(3): The granting of the proposal will provide for a facility that is consistent with the overall needs of the community.

The existing 2.5 MG Bolton Reservoir is a concrete slab-on-grade structure with 2:1 (horizontal: vertical) side slopes, constructed in 1915. An interior liner was installed in 1989 to try to keep the tank water tight and a Hypalon cover was placed over the reservoir in 1995 to protect water quality. The existing Bolton Pump Station was constructed in 1999 to pump to the Horton Reservoir and serve higher elevations in the Horton pressure zone. While the reservoir structure has served as the "hub" of the City's water system for nearly 100 years, it has safety, operational and maintenance issues that need to be addressed. The primary concerns are that it does not meet current seismic codes and would be unstable under the design-level earthquake. Prior inspections of the reservoir showed concrete spalling and some localized cracking. The floating cover appears to be reaching the end of its service life based on inspection and repairs of holes and tears in the cover in 2008, and extensive repairs again in 2012. Approximately 0.5 MG of the total 2.5 MG volume of water in the existing Bolton Reservoir is unusable since it cannot be removed due to the low elevation of the reservoir floor relative to the higher elevation of the reservoir inlet/outlet piping. Also, the configuration of the on-site piping allows the reservoir water to be short-circuited and by-passed, creating in-tank water quality issues as the Bolton Pump Station draws water directly from the supply main, by-passing the reservoir.

The WSMP recommended construction of improvements to address system wide storage deficits within the City. The City opted to proceed with the WSMP system wide storage alternative "Approach B – Storage and Emergency Supply Improvement", which includes the 4.0 MG Bolton Reservoir replacement. Pressure zones have storage needs that must be met at the pressure zone directly, such as fire suppression and operational needs, as well as an emergency storage component that can be aggregated system-wide. The City's WSMP states (on pages 6-14,6-15):

> *"Bolton Reservoir Replacement:* Construction of a new ground level reservoir to replace the existing Bolton Reservoir would address the current issues with the long-term maintenance of the Bolton Reservoir as well as the 0.8 mg deficit in the Willamette pressure zone and the 0.4 mg deficit in the Robinwood pressure zone. The capacity of the Bolton Reservoir Replacement will depend on a number of factors as previously discussed. For the purposes of this analysis, it is recommended that a 4.0 mg reservoir be constructed to replace the existing Bolton Reservoir. This reservoir volume provides replacement capacity for the existing Bolton Reservoir of 2.0 mg, addresses the combined storage deficit of the Willamette and Robinwood

> pressure zones of 1.2 mg and provides an additional 0.8 mg of storage to offset emergency supply needs."

The Bolton Reservoir is the only storage reservoir in the Bolton pressure zone. The reservoir provides for water service, fire suppression capacity and emergency water storage. Because of how the water system is operated, the Bolton Reservoir acts as a hub for the other pressure zones. The recommended 4.0 MG storage volume will meet the needs of the Bolton pressure zone which the reservoir directly serves, as well as eliminating existing storage deficiencies in the Willamette and Robinwood pressure zones, which can also be efficiently served from the proposed reservoir site. The proposed 4.0 MG reservoir at the existing Bolton Reservoir site would provide a lower total cost of storage when compared to the alternative of acquiring sites and building individual facilities for each pressure zone.

The proposed reservoir configuration will also allow for more usable storage to be available to pump to the upper pressure zones in the event of an operational disruption or emergency. The new on-site piping configuration will improve drinking water quality. Proposed site and foundation improvements will allow for a new reservoir that meets the current structural design codes for an essential facility, as well as improve the slope stability of the northern slope.

The existing Bolton Pump Station will have a new roof installed as part of the proposed project to address maintenance issues of the existing flat "bunker style" roof. The replacement roof will be a pitched standing seam metal roof with gabled ends to provide a building aesthetic more in keeping with the neighborhood character.

The proposed 4.0 MG reservoir is consistent with the WSMP which determined future water supply requirements and recommended improvements that correct existing system deficiencies and provide for future system needs under the approximate twenty (20) year planning period through 2030. The WSMP study area included the City's existing water service area and all areas within the City's existing Urban Growth Boundary (UGB). The City's water service area includes all areas within the City's current City limits, as well as a few homes outside the UGB that receive extra-territorial water service from the City. The proposed 4.0 MG reservoir will provide water service only to lands inside the 2008 WSMP study area, and will not introduce an urban service to new customers outside the UGB. The development of the proposed reservoir will not result in pressure for conversion of non-urbanizable lands to more intense uses.

60.070(A)(4): Adequate public facilities will be available to provide service to the property at the time of occupancy.

The existing Bolton Reservoir and pump station site currently has adequate public services to provide for fire protection, police, emergency medical services, roads, storm water drainage and water service. The proposed reservoir will be an unattended facility with an access driveway and parking area adequate to accommodate service vehicles. The driveway and parking area at the site will be constructed from an access point on Skyline Drive for maintenance vehicle access. As an unattended facility, the proposed reservoir will require minimal traffic for routine maintenance activities. It is anticipated that following construction, the facility may generate one vehicle trip per week on average, similar to the existing reservoir and pump station. The proposed reservoir will have on-site stormwater drainage facilities that will be connected to existing City infrastructure. The proposed on-site stormwater drainage facilities will not require new off-site infrastructure. The proposed reservoir will use currently available on-site water service for reservoir maintenance purposes only. The reservoir will not generate wastewater and therefore will not require sanitary sewer service. The proposed reservoir will not generate solid waste. No additional public facilities are required to serve the property to accommodate the proposed project.

60.070(A)(5): The applicable requirements of the zone are met, except as modified by this chapter.

The proposed site is an existing municipal water storage reservoir and pump station site located at 6111 Skyline Drive, on property zoned R-10 (Single-Family Residential Detached) as defined in CDC Chapter 11. A water storage tank (reservoir) is a major utility as defined in CDC Chapter 2, Definitions, in which water storage tank is included in the list of examples for the definition of "Utility, major". The proposed reservoir, as a major utility, is an allowed conditional use in the R-10 zone as described in CDC 11.060, Conditional Uses, which identifies "9. Utilities, major" as a conditional use allowed in the zoning district. The following further addresses how the applicable requirements of zone R-10 are met by the proposed use.

11.070 DIMENSIONAL REQUIREMENTS, USES PERMITTED OUTRIGHT AND USES PERMITTED UNDER PRESCRIBED CONDITIONS

Except as may be otherwise provided by the provisions of this code, the following are the requirements for uses within this zone:

1. The minimum lot size shall be 10,000 square feet for a single-family detached unit.

This provision is met. The lot size is 140,700 square feet.

2. The minimum front lot line length or the minimum lot width at the front lot line shall be 35 feet.

This provision is met. The front lot line length is approximately 350 feet.

3. The average minimum lot width shall be 50 feet.

This provision is met. The lot is approximately 350 feet by 400 feet.

4. *Repealed by Ord. 1622.*

5. Except as specified in CDC 25.070(C)(1) through (4) for the Willamette Historic District, the minimum yard dimensions or minimum building setback area from the lot line shall be:

- **a. For the front yard, 20 feet; except for steeply sloped lots where the provisions of CDC 41.010 shall apply.**
- **b. For an interior side yard, seven and one-half feet.**
- **c. For a side yard abutting a street, 15 feet.**
- **d. For a rear yard, 20 feet.**

The site in not in the Willamette Historic District. The minimum setback to either the reservoir or the pump station are met, as summarized below:

Setback Description	Required Minimum	Proposed Setback
Front	20 feet	98 feet
Side	7.5 feet	20 feet
Rear	20 feet	36 feet

6. The maximum building height shall be 35 feet, except for steeply sloped lots in which case the provisions of Chapter 41 CDC shall apply.

This provision is met. Building height is approximately 15 feet for the proposed reservoir and 19 feet for the proposed pump station after the roof replacement.

7. The maximum lot coverage shall be 35 percent.

This provision is met. The reservoir foot print is approximately 23,000 square feet. The existing pump station foot print is approximately 1,500 square feet with another 300 square feet for the generator. The total area is then 24,300 square feet in a 140,700 square foot lot. This results in an approximately lot coverage of 17 percent.

8. The minimum width of an accessway to a lot which does not abut a street or a flag lot shall be 15 feet.

This provision is not applicable. The lot abuts Skyline Drive and is not a flag lot.

9. The floor area ratio shall be 0.45. Type I and II lands shall not be counted toward lot area when determining allowable floor area ratio, except that a minimum floor area ratio of 0.30 shall be allowed regardless of the classification of lands within the property. That 30 percent shall be based upon the entire property including Type I and II lands. Existing residences in excess of this standard may be replaced to their prior

dimensions when damaged without the requirement that the homeowner obtain a non-conforming structures permit under Chapter 66 CDC.

This provision is met. The occupied space includes the pump station building at 1,500 square feet. Using the lot size of 140,700 square feet, the resulting floor area ratio is 0.010. When considering only the land with slopes less than 15 percent, the resulting floor area ratio is 0.013.

10. The sidewall provisions of Chapter 43 CDC shall apply. (Ord. 1175, 1986; Ord. 1298, 1991; Ord. 1377, 1995; Ord. 1538, 2006; Ord. 1614 § 2, 2013; Ord. 1622 § 24, 2014)

This provision is not applicable. Per CDC 43.020, the provisions of Chapter 43 "shall apply to all new home construction and remodels." No homes are associated with this project.

11.080 DIMENSIONAL REQUIREMENTS, CONDITIONAL USES

Except as may otherwise be established by this code, the appropriate lot or parcel size for a conditional use shall be determined by the approval authority at the time of consideration of the application based upon the criteria set forth in CDC 60.070(A) and (B). (Ord. 1636 § 9, 2014)

As this is a conditional use, the dimension requirements (11.080) are developed from the criteria set out in 60.070(A) and (B). The Site Plan (Figure 1) in the Appendices and the responses to 60.070(A) and (B) show that the parcel size is appropriate for the intended use.

11.090 OTHER APPLICABLE DEVELOPMENT STANDARDS
A. The following standards apply to all development including permitted uses:

1. Chapter 34 CDC, Accessory Structures, Accessory Dwelling Units, and Accessory Uses.

The existing pump station and proposed reservoir are defined in CDC Chapter 2 as "Utility, major" and are conditional uses per CDC 11.060.

As part of the proposed project, the existing storage building will need to be demolished and removed. A location for a future replacement storage building has been identified in the southwest corner of the site. This future structure will be an Accessory Structure as defined by CDC 34.020, but will not be constructed as part of the proposed project.

As shown on the Site Plan in the Appendices, the location of the future replacement storage building will meet the required setbacks of CDC 11.070(5) as shown below.

Setback Description	Required Minimum	Proposed Setback
Front	20 feet	40 feet
Side	7.5 feet	15 feet
Rear	20 feet	300 feet

The future replacement storage building is anticipated to be a wood-framed, single story structure with an appearance similar to existing residential properties in the neighborhood.

2. Chapter 35 CDC, Temporary Structures and Uses.

These provisions are met. A temporary portable job trailer may be used at this site by the construction contractor. Per CDC 35 per 35.020(A), "Construction related uses including, but not limited to, trailers and staging areas," are exempt structures. No other temporary structures are included in this project.

3. Chapter 38 CDC, Additional Yard Area Required; Exceptions to Yard Requirements; Storage in Yards; Projections into Yards.

These provisions are met. The reservoir structure and pump station structure will both be more than three feet from the property line, more than 25 feet from the nearest street, will not have any projections extending into the front or rear yard (such as porches, decks or balconies) by more than five feet. The front yard of the property will not be used for storage of vehicles or trailers.

4. Chapter 40 CDC, Building Height Limitations, Exceptions.

Repealed by Ordinance 1604.

5. Chapter 41 CDC, Structures on Steep Lots, Exceptions.

These provisions are met. The maximum building height is 35 feet. The proposed reservoir will be approximately 15 feet in height as measured from the lowest point of grading 5 feet from the structure to the top of the roof. The proposed pump station replacement roof will be approximately 19 feet in height.

6. Chapter 42 CDC, Clear Vision Areas.

These provisions are met. The clear vision area for the 24-foot wide driveway as computed per CDC 42.040 are met. The clear vision area is illustrated on the Site Plan (Figure 1) in the Appendices.

7. Chapter 44 CDC, Fences.

These provisions are met.

A new security fence six feet in height will be installed along the side and back yard property lines within the required side and back yards. The fence will be a black coated chain link fence that will not obscure vision.

A decorative metal security fence six feet in height will also be installed along the front of the property outside of the front yard setback distance. The fence will meet the clear vision requirements of CDC 42.

8. Chapter 46 CDC, Off-Street Parking, Loading and Reservoir Areas.

These provisions are met.

a. **46.020:** Parking for employee access will continue to use the existing paved area in front of the pump station.

b. **46.090:** It is anticipated that no parking spaces will be required to meet CDC 46.090. The building and site use is not intended for occupied use. However, up to two (2) staff may routinely visit the site as part of routine operations. As the site is not open to the public, no handicap parking spaces will be provided. The existing paved area in immediately in front of the pump station can accommodate a minimum of two parking spaces. There is ample paved and gravel areas to accommodate additional parking if needed.

c. **46.130:** The proposed use does not receive or distribute material or merchandise requiring off-street loading spaces.

d. No parking will be provided for the public.

e. No bicycle facilities will be placed on this site.

9. Chapter 48 CDC, Access, Egress and Circulation.

These provisions are met.

a. **48.030(E)(3-6):** The vertical clearance will exceed 13 feet, six inches; the site has adequate paved fire apparatus access meeting the Oregon Fire Code 503.1.1.

b. **48.040:** The driveway will be a 24-foot width accommodating two-way traffic.

c. **48.060:** The access curb cut will be more than 150 feet from any intersection and more than 150 feet from the adjacent property driveway.

10. Chapter 52 CDC, Signs.

There will be no signs on the site with the proposed use, therefore Chapter 52 is not applicable.

11. Chapter 54 CDC, Landscaping.

These provisions are met as described below. See also the responses to CDC 60.070(A)(6)(c).

a. **54.020(D):** There are no heritage trees identified on the project site.

b. **54.020(E)(2):** More than 20 percent of the site is required to be landscaped. Approximately 57 percent of the site will be covered with existing trees, proposed vegetative screening or ground cover, as shown in the proposed Landscape Plan included in the Appendices.

c. **54.020(E)(3)(a):** All exposed ground not otherwise landscaped will be covered in drought tolerant grasses. Tree preservation is consistent with the Arborist Report included in the Appendices.

d. **54.020(E)(3)(i):** The pump station parking area will be screened and buffered from neighboring residential properties

e. **54.020(E)(3)(l):** Proposed trees were selected with characteristics in accordance with this provision, including native, drought-tolerant species that provide a spreading canopy without sticky leaves, dripping sap, seed pods or bearing fruit.

f. **54.020(G): Water Resource Area (WRA):** The project site is not in a WRA.

g. **54.050: Installation:** All landscaping to be added to the site will comply with the requirements of installation as laid out in this section.

B. The provisions of Chapter 55 CDC, Design Review, apply to all uses except detached single-family dwellings, residential homes and residential facilities. (Ord. 1590 § 1, 2009)

The provisions of Chapter 55 are addressed in Section IV of this application, “Class II Design Review Narrative”.

60.070(A)(6): The supplementary requirements set forth in Chapters 52 to 55 CDC, if applicable, are met.

A. Chapter 52 Signs

There will be no signs on the site with the proposed use, therefore Chapter 52 is not applicable.

B. Chapter 53 Sidewalk Use

There will be no sidewalks constructed with the proposed use, therefore Chapter 53 is not applicable.

C. Chapter 54 Landscaping

54.020 APPROVAL CRITERIA

A. Every development proposal requires inventorying existing site conditions which include trees and landscaping. In designing the new project, every reasonable attempt should be made to preserve and protect existing trees and to incorporate them into the new landscape plan. Similarly, significant landscaping (e.g., bushes, shrubs) should be integrated. The rationale is that saving a 30-foot-tall mature tree helps maintain the continuity of the site, they are qualitatively superior to two or three two-inch caliper street trees, they provide immediate micro-climate benefits (e.g., shade), they soften views of the street, and they can increase the attractiveness, marketability, and value of the development.

The Arborist Report in the Appendices includes the tree inventory. The proposed reservoir location is as far from Skyline Drive as can be situated per the Geotechnical Engineer's recommendations to provide for adequate slope stability and reservoir structure seismic performance. During construction, shoring will be employed to minimize the removal of trees in the tree cluster located at the southeast property corner.

B. To encourage tree preservation, the parking requirement may be reduced by one space for every significant tree that is preserved in the parking lot area for a maximum reduction of 10 percent of the required parking. The City Parks Supervisor or Arborist shall determine the significance of the tree and/or landscaping to determine eligibility for these reductions.

The parking requirements do not result in the removal of any existing trees. The incentive in 54.020(B) is not applicable.

C. Developers must also comply with the municipal code chapter on tree protection.

The construction contract documents will include the requirement that the construction contractor comply with the municipal code chapter on tree protection.

D. Heritage trees. Heritage trees are trees which, because of their age, type, notability, or historical association, are of special importance. Heritage trees are trees designated by the City Council following review of a nomination. A heritage tree may not be removed without a public hearing at least 30 days prior to the proposed date of removal. Development proposals involving land with heritage tree(s) shall be required to protect and save the tree(s). Further discussion of heritage trees is found in the municipal code.

There are no heritage trees identified within the project area, therefore CDC 54.020(D) is not applicable.

E. Landscaping – By type, location and amount.
1. Residential uses (non-single-family). A minimum of 25 percent of the gross area including parking, loading and service areas shall be landscaped, and may include the open space and recreation area requirements under CDC 55.100. Parking lot landscaping may be counted in the percentage.

This provision is not applicable as the proposed use is not residential.

2. Non-residential uses. A minimum of 20 percent of the gross site area shall be landscaped. Parking lot landscaping may be counted in the percentage.

This provision is met. Approximately 57 percent of the site will be vegetated.

3. All uses (residential uses (non-single-family) and non-residential uses):
a. The landscaping shall be located in defined landscaped areas which are uniformly distributed throughout the parking or loading area. There shall be one shade tree planted for every eight parking spaces. These trees shall be evenly distributed throughout the parking lot to provide shade. Parking lots with over 20 spaces shall have a minimum 10 percent of the interior of the parking lot devoted to landscaping. Pedestrian walkways in the landscaped areas are not to be counted in the percentage. The perimeter landscaping, explained in subsection (E)(3)(d) of this section, shall not be included in the 10 percent figure. Parking lots with 10 to 20 spaces shall have a minimum five percent of the interior of the parking lot devoted to landscaping. The perimeter landscaping, as explained above, shall not be included in the five percent. Parking lots with fewer than 10 spaces shall have the standard perimeter landscaping and at least two shade trees. Non-residential parking

areas paved with a permeable parking surface may reduce the required minimum interior landscaping by one-third for the area with the permeable parking surface only.

No formal parking area is required for the proposed use, therefore these provisions are not applicable. Refer to the response for CDC 46.090.

b. The landscaped areas shall not have a width of less than five feet.

The landscaped areas will be at least 10 feet in the smallest dimension, so this provision is met.

c. The soils, site, proposed soil amendments, and proposed irrigation system shall be appropriate for the healthy and long-term maintenance of the proposed plant species.

The soil preparation and soil amendments will be specified by a licensed Landscape Architect to be appropriate for establishment and long-term maintenance of the proposed plant species. This provision is met.

d. A parking, loading, or service area which abuts a street shall be set back from the right-of-way line by perimeter landscaping in the form of a landscaped strip at least 10 feet in width. When a parking, loading, or service area or driveway is contiguous to an adjoining lot or parcel, there shall be an intervening five-foot-wide landscape strip.

No parking, loading or service area will abut a street or be contiguous to and adjoining lot, therefore this provision is met.

e. If over 50 percent of the lineal frontage of the main street or arterial adjacent to the development site comprises parking lot, the landscape strip between the right-of-way and parking lot shall be increased to 15 feet in width and shall include terrain variations (e.g., one-foot-high berm) plus landscaping. This extra requirement only applies to one street frontage.

No parking lot will be located along the frontage, therefore this provision is not applicable.

f. A parking, loading, or service area which abuts a property line shall be separated from the property line by a landscaped area at least five feet in width and which shall act as a screen and noise buffer, and the adequacy of the screen and buffer shall be determined by the criteria set forth in CDC 55.100(C) and (D), except where shared parking is approved under CDC 46.050.

No parking, loading or service areas will abut a property line. Screening and buffering exceeding five feet in width will be established along the side and back property lines. This provision is met.

g. All areas in a parking lot not used for parking, maneuvering, or circulation shall be landscaped.

The area surrounding the parking and maneuvering pad in front of the pump station is landscaped.

h. The landscaping in parking areas shall not obstruct lines of sight for safe traffic operation.

No landscaping will be installed which would obstruct access and maneuvering along paved and gravel access roads on the proposed site, therefore this provision is met.

i. Outdoor storage areas, service areas (loading docks, refuse deposits, and delivery areas), and above-ground utility facilities shall be buffered and screened to obscure their view from adjoining properties and to reduce noise levels to acceptable levels at the property line. The adequacy of the buffer and screening shall be determined by the criteria set forth in CDC 55.100(C)(1).

The site will be screened and buffered along the side and rear yards along the property line as shown on the Landscape Plan in the Appendices. No noise is associated with operation of the reservoir. This provision is met.

j. Crime prevention shall be considered and plant materials shall not be located in a manner which prohibits surveillance of public and semi-public areas (shared or common areas).

This provision is met. The plantings along the frontage will not obstruct the view of the site from Skyline Drive. Refer to the Landscape Plan in the Appendices.

k. Irrigation facilities shall be located so that landscaped areas can be properly maintained and so that the facilities do not interfere with vehicular or pedestrian circulation.

No permanent irrigation facilities are associated with the proposed landscaping, therefore this provision is not applicable.

l. For commercial, office, multi-family, and other sites, the developer shall select trees that possess the following characteristics:

1) **Provide generous "spreading" canopy for shade.**
2) **Roots do not break up adjacent paving.**
3) **Tree canopy spread starts at least six feet up from grade in, or adjacent to, parking lots, roads, or sidewalks unless the tree is columnar in nature.**
4) **No sticky leaves or sap-dripping trees (no honey-dew excretion).**
5) **No seed pods or fruit-bearing trees (flowering trees are acceptable).**
6) **Disease-resistant.**
7) **Compatible with planter size.**
8) **Drought-tolerant unless irrigation is provided.**
9) **Attractive foliage or form all seasons.**

The selected tree species meet these provisions. Refer to the planting list.

m. Plant materials (shrubs, ground cover, etc.) shall be selected for their appropriateness to the site, drought tolerance, year-round greenery and coverage, staggered flowering periods, and avoidance of nuisance plants (Scotch broom, etc.).

The selected plant materials meet these provisions. Refer to the planting list.

F. <u>Landscaping (trees) in new subdivision</u>.

The proposed development is not a subdivision, therefore the provisions of 54.020(F) are not applicable.

G. <u>Landscaping requirements in water resource areas (WRAs)</u>. Pursuant to CDC <u>32.110</u>(E)(3) the requirements of this chapter relating to total site landscaping, landscaping buffers, landscaping around parking lots, and landscaping the parking lot interior may be waived or reduced in a WRA application without a variance being required. (Ord. 1408, 1998; Ord. 1463, 2000; Ord. 1623 § 5, 2014; Ord. 1636 § 36, 2014)

No waiver is requested, therefore this provision is not applicable.

54.030 PLANTING STRIPS FOR MODIFIED AND NEW STREETS

All proposed changes in width in a public street right-of-way or any proposed street improvement shall, where feasible, include allowances for planting strips. Plans and specifications for planting such areas shall be integrated into the general plan of street improvements. This chapter requires any multi-family, commercial, or public facility which causes change in public right-of-way or street improvement to comply with the street tree planting plan and standards.

The proposed development does not include any changes to the public street.

D. Chapter 55 Design Review

Response to approval criteria for CDC Chapter 55 is provided in Section IV of this document, "Class II Design Review Narrative".

60.070(A)(7): The use will comply with the applicable policies of the Comprehensive Plan.

<u>Goal 1: Citizen Involvement</u>

The proposed use for a replacement water storage reservoir at this site is consistent with the applicable policies of Goal 1. Reponses are provided below to document that the proposal is consistent with the applicable policies.

Policy 1-4: Provide timely and adequate notice of proposed land use matters to the public to ensure that all citizens have an opportunity to be heard on issues and actions that affect them.

The Community Development Code includes provisions to meet Policy 1-4. These provisions were met the through the neighborhood contact process in accordance with CDC 99.038, which included neighborhood notices of land use review, and invitations to the neighborhood meeting conducted on April 23, 2015. The City will provide a noticed public hearing before making a final decision on the applications, in accordance with CDC 99.

The city website includes information and updates on the reservoir replacement project and provides contact information for appropriate City staff.

Policy 1-5: Communicate with citizens through a variety of print and broadcast media early in and throughout the decision-making process.

The applicant has used the mailing and public meeting process outlined in CDC 99.038. The applicant has also posted information about the project on the City website and provided information for newspaper articles in the West Linn Tidings. In addition, the applicant has contacted and met with adjacent property owners in advance of and following the neighborhood meeting process.

<u>Goal 2: Land Use Planning</u>

The proposed use for a replacement water storage reservoir at this site is consistent with the applicable policies of Goal 2. Reponses are provided below to document that the proposal is consistent with the applicable policies.

Policy 2-1: Maintain effective coordination with other local governments, special districts, state and federal agencies, Metro, the West Linn-Wilsonville School District, and other governmental and quasi-public organizations.

The timing and schedule for the reservoir replacement and the City's planned interim operations during construction have been coordinated with the City of Oregon City and the SFWB regarding the reservoir shutdown; the City of Lake Oswego regarding the timing of improvements at the City's Emergency Intertie Pump Station to be made in advance of the reservoir shutdown; and the Lake Oswego-Tigard Partnership regarding the timing of the water treatment plant improvements and new water supply availability relative to the reservoir shutdown schedule.

Goal 5: Open Spaces, Scenic and Historic Areas, and Natural Resources

The proposed use for a replacement water storage reservoir at this site is consistent with the applicable policies of Goal 5. The City-owned site is across the street from Wilderness Park and has mature trees on the north and south edges of the property, away from the existing reservoir, pump stations and storage building. The central location of the proposed reservoir and recommended slope grading improvements requires the removal of approximately 81 non-significant, 28 significant trees, and an area heavily infested with invasive plants including English ivy and blackberry. The overall nature of the site will be retained and preserved with the proposed replacement reservoir project and associated landscape restoration and reforestation. Reponses are provided below to document that the proposal is consistent with the applicable policies.

Policy 5-2: Where appropriate, require the planting of trees as a condition of approval for any land development proposal, consistent with the City's street tree ordinance and recommendations of the City Arborist.

Tree planting will be consistent with the City's street tree ordinance and recommendations of the City Arborist. The trees proposed for planting as landscaping and screening along the side and back property lines include Incense Cedar, Douglas-fir and Western Red Cedar. Vine Maples and assorted native shrubs are proposed for planting under the existing Douglas-fir stand to the south and southeast of the reservoir, to replace the non-native grass with an appropriate understory to improve wildlife habitat.

Policy 5-3: Provide buffer areas around heritage trees, significant trees, and tree clusters to ensure their preservation.

The stand of Douglas-fir trees along Skyline Drive includes the significant trees to be preserved. A buffer area will be provided during construction in accordance with CDC 55.100(B) and as recommended in the Arborist Report included in the Appendices.

Policy 5-4: Require that areas containing tree clusters, significant trees, and native vegetation along natural drainage courses and waterways in areas of new development be maintained to the maximum extent possible to preserve habitats, prevent erosion, and maintain water quality.

This policy is implemented by the CDC in Chapter 54, Landscaping; Chapter 55, Design Review; and is addressed in the City's Tree Technical Manual. This project meets the requirements of the CDC.

Tight excavation shoring will be used for the proposed reservoir excavation and construction to minimize tree removal to the south and southeast of the proposed reservoir location. The tree stand containing significant trees in the southeast corner of the property will be maintained to the maximum extent possible. The excavation shoring will preserve approximately 5 significant trees. Areas northeast of the reservoir will require tree removal where mass excavation and slope grading improvements are required. Some clearing and tree removal will be needed in the utility easements north of the City owned property to complete the proposed piping replacement. The area of new development does not include natural drainage courses and waterways.

Policy 5-5: Preserve important wildlife habitat by requiring clustered development or less dense zoning in areas with wetlands and riparian areas, natural drainage ways, and significant trees and tree clusters.

The tree stand containing significant trees in the southeast corner of the property will be maintained to the maximum extent possible. Tight excavation shoring will be used for the proposed reservoir excavation and construction to minimize tree removal to the south and southeast of the proposed reservoir location. Vine Maples and assorted native shrubs are proposed for planting under the existing Douglas-fir stand to the south and southeast of the reservoir, to replace the non-native grass with an appropriate understory to improve wildlife habitat. The project site does not contain wetlands, riparian areas and natural drainage ways.

Policy 5-6: Restore, enhance, and expand the existing habitats found along rivers and streams, including planting native trees to reduce water temperatures.

The project site is not located along a river or stream.

Policy 5-7: Enhance and expand vegetation, particularly native species, on hillsides and in natural areas to prevent erosion and improve wildlife habitat.

As part of the proposed project, the steep slopes along the northern property line will be excavated to remove fill and invasive plant species including English ivy and blackberry, then regraded to reduce slopes. The newly graded slopes will be revegetated with drought tolerant native species to prevent erosion and improve wildlife habitat.

Policy 5-8: Require and enforce erosion control standards for new development.

The project will meet City of West Linn erosion control standards. The project will include an Oregon DEQ 1200-C permit administered by the Clackamas County Water Environment Services.

Policy 9: Maintain and improve existing storm water detention and treatment standards to ensure that the impact of new development does not degrade water quality and wildlife habitat.

The stormwater management approach will maintain the existing peak runoff rate and will not degrade water quality. The Stormwater Management Plan will be prepared by a licensed professional engineer and reviewed by the City of West Linn Engineering Department. The preliminary Stormwater Management Plan is included in the Appendices.

Goal 6: Air, Water & Land Resources Quality

The proposed use for a replacement water storage reservoir at this site is consistent with the applicable policies of Goal 6. The proposed replacement water storage reservoir will not involve generation or emission of noise, refuse, water pollution or impacts to air quality in any way. The project will comply with all applicable state and federal environmental standards. The construction contractor will be required to obtain an Oregon DEQ National Pollutant Discharge Elimination System (NPDES) Stormwater Discharge Permit #1200-C., and will develop and implement the final Erosion and Sediment Control Plan (ESCP) as required by the permit. Reponses are provided below to document that the proposal is consistent with the applicable policies.

Policy 6-1: Require that new development be designed and constructed to prevent degradation of surface and groundwater quality by runoff.

Consistency with this policy will be met through the proposed project's compliance with the City of West Linn stormwater design and construction standards, and through the proposal's consistency with the applicable policies of Goal 5 described above.

Specifically, the proposed stormwater management approach will reduce the peak stormwater runoff to predevelopment conditions and provide water quality treatment to the channelized runoff to Bolton Creek. Currently, the reservoir cover runoff is untreated.

Policy 6-2: Require that City construction projects, maintenance activities, and operating procedures be designed and operated so as to not degrade surface or ground water quality.

Consistency with this policy will be met through the proposed project's compliance with the City of West Linn stormwater design and construction standards, compliance with the City of West Linn stormwater facility operation and maintenance agreements, and through the proposal's consistency with the applicable policies of Goal 5 described above.

In addition to the stormwater management approach discussed in the Policy 6-1 comments, an operation and maintenance agreement will be established to ensure long-term efficacy of the stormwater facilities.

Policy 6-5: Where feasible, use open, naturally vegetated drainageways to reduce stormwater runoff and improve water quality

Vegetated stormwater detention and water quality facilities will be constructed and planted with native species to reduce stormwater runoff rates and improve water quality. The proposed stormwater facilities will be lined with impermeable materials to prevent on-site infiltration of stormwater runoff in accordance with recommendations of the geotechnical engineer. The facilities will include leak detection monitoring systems to confirm liner integrity.

Policy 6-6: Meet the goals of Title 3 of the Metro Urban Growth Management Functional Plan.

Title 3 outlines measures "to protect the beneficial water uses and functions and values of resources within the Water Quality and Flood Management Areas by limiting or mitigating the impact of these areas from development activities and protecting life and property from dangers associated with flooding." Measures include amendment of city comprehensive plans and implementing ordinances consistent with Title 3 performance standards. The performance standards include those for flood management, water quality, erosion and sediment control, implementation tools, and map administration.

This project meets the goals for each performance standard as follows:

- The flood management goal is met by complying with the adopted stormwater management guideline which restricts developed peak runoff to the pre-development rate.
- The water quality goal is met by complying with the adopted stormwater management guidelines for surface water treatment. This project will exceed the minimum standard by treating runoff from both the required impervious area as well as the collected runoff from pervious areas.
- The erosion and sediment control goal is met by complying with the city standards for erosion control, using best management practices, and obtaining and complying with a Oregon DEQ 1200-C permit.

The implementation and map administration goals pertain to the cities and are not applicable to this project.

Policy 6-7: Require up to date erosion control plans for all construction and actively enforce applicable City codes and regulations.

Anticipated Erosion Control Measures (Figure 3) are include in the Appendices. The project will meet City of West Linn erosion control standards. The project will include an Oregon DEQ 1200-C permit administered by the Clackamas County Water Environment Services.

Policy 6-8: Encourage the use of alternative permeable materials for construction of parking areas to reduce stormwater runoff and improve water quality.

The vehicle access areas will be gravel outside of the paved driveway leading to the pump station. The paved access is required for fire apparatus (truck) access and will provide improved access for infrequent pump station maintenance vehicles and staff vehicle access.

Goal 7: Areas Subject to Natural Disasters and Hazards

The proposed use for a replacement water storage reservoir at this site is consistent with the applicable policies of Goal 7. Reponses are provided below to document that the proposal is consistent with the applicable policies.

State Planning Goal 7 prohibits locating developments subject to damage or loss of life in known areas of natural disasters and hazards without appropriate safeguards. Plans are to be based on an inventory of known areas of natural disaster and hazards, including flooding, erosion, landslides, earthquakes, weak foundation soils, or other hazards that may be unique to local or regional areas.

Policy 7-1: Require development and associated alterations to the surrounding land to be directed away from hazardous areas.

The City's Natural Hazard Mitigation Plan, Map 16 Potential Landslides included in the Appendices identifies "DOGAMI Potential Landslides" and areas with slopes that exceed 25 percent. The proposed reservoir site is not within the areas identified as DOGAMI Potential Landslides on Map 16, but it does have slopes within the property limits that exceed 25 percent as shown the attached Site Analysis Map.

The Natural Hazard Mitigation Plan, Map 17 Landslide Vulnerability Analysis included in the Appendices, identifies all areas with slopes that exceed 25 percent as "Landslide Hazard Areas". The existing reservoir is identified as one of the "Assets and Infrastructure within Landslide Area" on Map 17. The attached Site Analysis Map shows areas within the site with slopes that exceed 25 percent. As part of recent preliminary engineering, geotechnical investigation and analysis was performed which confirmed the geotechnical suitability of the site, with recommendations to address potential hazards related to steep slopes identified on Map 17.

The geotechnical investigation and geologic analysis completed to confirm the geotechnical suitability of the existing site included a review of available pertinent information and historical data for the site and vicinity, site and vicinity reconnaissance, additional soil borings and laboratory testing, slope stability analysis, an evaluation of the large ancient landslide that extends across the City's northern slope, and a site-specific seismic hazard study that is required by the State Structural Code for all essential facilities. The geotechnical work was performed by two geotechnical firms, GRI and Cornforth Consultants.

The report by GRI titled "Geotechnical Investigation and Site-Specific Seismic Hazard Study 4-MG Bolton Reservoir West Linn, Oregon", documents the work accomplished and provides conclusions and recommendations for founding the proposed reservoir on the site. The report by Cornforth Consultants tiled "Geotechnical Engineering Services, Bolton Reservoir Seismic Landslide Evaluation, West Linn, Oregon", documents their geologic reconnaissance and qualitative seismic stability evaluation of the ancient landslide surrounding the existing Bolton Reservoir. Both reports are included in the Appendices.

A very small shallow flow slide was reported to have occurred in the 1970's near the northeast corner of the Bolton Reservoir site, with likely causes related to a water main break and fill that been placed at the top of the slope during the original reservoir construction. The geotechnical findings reported that the existing reservoir structure, as it stands now near the top of the slope, is at risk of instability and failure under the design-level earthquake based on the slope stability analysis. Recommendations for the proposed reservoir include removal of the soil at top of slope, setting the new reservoir back from slope, providing adequate foundation drainage and ground improvement (such as aggregate piers) below the floor slab to achieve a satisfactory seismic factor of safety against local instability.

One of the key reasons the proposed reservoir has a much higher safety factor than the existing reservoir is the overall net decrease in loads on the site with the new project. While the new reservoir will have a larger volume of stored water, the additional stored water will replace heavier soil on the site. This combined with removing a large amount of fill from the top of the slope will actually reduce loading or weight on the site by approximately 6,000 tons, greatly increasing the site's safety. The planned ground improvement beneath the reservoir, removal of soil at the top of the slope along the north side of the site, and the gravel pad and sub-drainage system around and beneath the reservoir will improve local factors of safety as they relate to potential reservoir instability. The proposed reservoir replacement will move the new structure away from the steep slopes and further reduce the risk of landslides associated with the steep slope.

Also, a review of recent updated mapping available from DOGAMI, SLIDO (Statewide Landslide Information Database of Oregon, Release 2, 2011) indicated that the existing reservoir and proposed project site were within a very large ancient landslide mapped by DOGAMI. This landslide is a prehistoric, deep-seated translational rock landslide referred to as Canby 133 that covers approximately 170 acres of West Linn's northern slopes. GRI reported that the mapped northeast boundary of the Canby 133 landslide near the site is essentially coincident with the prominent straight and abrupt topographic escarpment associated with the Bolton Fault. In their opinion, this indicates the Bolton Fault cross-cuts the toe of the Canby 133 landslide, and therefore, the Canby 133 landslide is likely on the order of at least 15,000 to 20,000 years old.

Geotechnical reconnaissance of the ancient landslide performed by both GRI and Cornforth Consultants did not identify signs of active movement, especially along the margins where differential movement would be greatest. The qualitative seismic stability evaluation of the ancient landslide included a study of observed performance of similar large, translational landslides subjected to earthquake motions. This included six case histories that document seismically-induced landslide displacements in slide masses with characteristics similar to the landslide surrounding the Bolton Reservoir, ranging from the local 1949 Olympia, WA - Magnitude 7.1 earthquake, to the more recent 2010 Chile – Magnitude 8.8 earthquake. One of the more important observations gained from the post-seismic reconnaissance was that most pre-existing landslides have remained relatively stable with small to moderate displacements during the seismic event. Another observation is that translational landslides tend to move as coherent masses with small to moderate differential movements away from the slide margins. One of the biggest risks to a water bearing structure is differential settlement across the foundation that could cause unbalanced forces on the structure or foundation failure. Based on the analysis, the ancient landslide around Bolton Reservoir is likely to move feet rather than tens of feet if it moves during a large earthquake. It is expected that structures located at the margins of the slide will be subjected to larger differential displacements, therefore the existing site located in the middle of the large ancient landslide mass is preferred.

Due to the large size of the ancient landslide and potential deep failure surfaces, mitigation measures to improve the stability of the large ancient landslide mass are likely not practical or cost effective. Based on the available information, the risk of significant movement of the large landslide within the design life of the reservoir is expected to be low and would most likely occur during/following a large seismic event. It is expected that if movement of the large landslide mass occurs, the ground supporting the reservoir will tend to “raft” along with the greater landslide mass and risk of significant differential movements beneath the reservoir will be reduced. In addition, the proposed ground improvement will strengthen the ground beneath the reservoir, which will further reduce the risk of significant differential movements. It is expected that the proposed structure can accommodate the anticipated level of movement and the rafting effect that may potentially accompany a large seismic event. Double ball, flexible expansion joints will be provided on all piping connections to the reservoir adjacent to the exterior edge of the reservoir’s foundation or wall to provide the piping with additional flexibility to prevent damage from potential seismic induced settlement or movement. Seismic response infrastructure will be installed on site. The reservoir outlet piping will be equipped with electric actuator and seismic sensing controller to isolate the reservoir when a seismic event threshold is detected. This will maintain water in the reservoir for emergency use, preventing unintentional draining of the tank should there be a large off-site water main break in the system.

Policy 7-2: Restrict development except where design and construction techniques can mitigate adverse effects.

This policy is implemented by the CDC in Chapter 55, Design Review, specifically 55.100(B)(4). This project meets the requirements of the CDC.

The proposed development will meet current International Building Code (IBC) requirements and the Oregon Structural Specialty Code design standards for essential facilities.

The proposed reservoir project will include removal of the soil at the top of slope, setting the new reservoir back from the slope, providing adequate foundation drainage and aggregate piers below the floor slab for foundation improvement. One of the key reasons the proposed reservoir has a much higher safety factor than the existing reservoir is the overall net decrease in loads on the site with the new project. While the new reservoir will have a larger volume of stored water, the additional stored water will replace heavier soil on the site. This combined with removing a large amount of fill from the top of the slope will actually reduce loading or weight on the site by approximately 6,000 tons, greatly increasing the site's safety. The planned ground improvement beneath the reservoir, removal of soil at the top of the slope along the north side of the site, and the gravel pad and sub-drainage system around and beneath the reservoir will improve local factors of safety as they relate to potential reservoir instability. The new reservoir as planned, will not adversely affect the existing slope stability. The proposed reservoir replacement will move the new structure away from the steep slopes and further reduce the risk of landslides associated with the steep slope.

In addition, the proposed ground improvement will strengthen the ground beneath the reservoir, which will further reduce the risk of significant differential movements. It is expected that the proposed structure can accommodate the anticipated level of movement and the rafting effect that may accompany potential movement during or following a large seismic event. Double ball, flexible expansion joints will be provided on all piping connections to the reservoir adjacent to the exterior edge of the reservoir's foundation or wall to provide the piping with additional flexibility to prevent damage from potential seismic induced settlement or movement. Seismic response infrastructure will be installed on site. The reservoir outlet piping will be equipped with electric actuator and seismic sensing controller to isolate the reservoir when a seismic event threshold is detected. This will maintain water in the reservoir for emergency use, preventing unintentional draining of the tank should there be a large off-site water main break in the system.

The other very important safety improvement is the reservoir structure itself, which will be a strand-wound, circular, prestressed concrete reservoir designed and constructed to AWWA D110, Type I standards. This reservoir type has cast-in-place concrete walls with vertical prestressed reinforcement. Vertical prestressing tendons are cast inside the structure's walls to provide compression that counteracts the effects of differential dryness and thermal loads. To provide an unrestrained connection and to reduce bending moments induced by hydrostatic, thermal, backfill and seismic forces on the tank wall, the roof and floor are separated from the corewall by neoprene bearing pads. This "free-sliding" connection at the wall base and wall top enhances the seismic performance of the tank by allowing the floor, wall and roof to act independently of each other. A continuous PVC bulb waterstop between the floor and wall assures full liquid tightness of the joint.

Policy 7-3: Require soils and geologic studies for development in hazardous areas.

This policy is implemented by the Municipal Code in Chapter 8, Building. Specifically 8.055 adopts the Oregon Structural Specialty Code. This project meets the requirements of the Municipal Code.

As the Oregon Structural Specialty Code identifies the drinking water reservoir as an essential facility, a Geotechnical Investigation and Site-Specific Seismic Hazard Study was performed to support design of the structure. The study characterizes the soils and analyses the site slope stability to include performance under seismic events. The report by GRI titled "Geotechnical Investigation and Site-Specific Seismic Hazard Study 4-MG Bolton Reservoir West Linn, Oregon", documents the work accomplished and provides conclusions and recommendations for founding the proposed reservoir on the site and is included in the Appendices. The report by Cornforth Consultants tiled "Geotechnical Engineering Services, Bolton Reservoir Seismic Landslide Evaluation, West Linn, Oregon", documents their geologic reconnaissance and qualitative seismic stability evaluation of the ancient landslide surrounding the existing Bolton Reservoir.

Policy 7-4: Promote slope and soil stability and the use of natural drainageways in areas with landslide potential by retaining existing vegetation in those areas to the greatest extent possible.

Most of the existing trees and vegetation at the top of the steep slope along the north side of the site cannot be retained due to the proposed slope improvements. The steep slopes along the northern property line will be excavated to remove fill, and regraded to reduce slopes as part of the proposed project, therefore existing trees, ground cover and invasive plant species will need to be removed to accomplish this work. The newly graded slopes will be revegetated and reforested with drought tolerant native species to prevent erosion, promote slope and soil stability.

The geotechnical findings reported that the existing reservoir structure, as it stands now near the top of the slope, is at risk of instability and failure under the design-level earthquake based on the slope stability analysis. The planned ground improvement beneath the reservoir, removal of soil at the top of the slope along the north side of the site, and the gravel pad and sub-drainage system around and beneath the reservoir will improve local factors of safety as they relate to potential reservoir instability. One of the key reasons the proposed reservoir has a much higher safety factor than the existing reservoir is the overall net decrease in loads on the site with the new project. The new reservoir and site restoration as planned, will not adversely affect the existing slope stability. The proposed reservoir replacement will move the new structure away from the steep slopes and further reduce the risk of landslides associated with the steep slope. The proposed slope revegetation and reforestation measures will promote slope and soil stability.

Policy 7-5: Follow state and regional designations and construction standards regarding earthquake hazards.

The proposed development will meet current International Building Code (IBC) requirements and the Oregon Structural Specialty Code (OSCC) design standards for essential facilities.

Policy 7-12: Refer to current seismic information during development review, including in the pre-application meeting, and when enacting new regulations governing the location of structures and land uses.

The City of West Linn uses the current State of Oregon building code and standards. A site-specific seismic hazard study was completed for the proposed reservoir project in accordance with the Oregon Structural Specialty Code (OSSC) design standards for essential facilities.

Goal 11: Public Facilities and Services

The proposed use for a replacement water storage reservoir at this site is consistent with the applicable policies of Goal 11. Goal 11 requires that public services be made available to areas in a manner so as to be adequate to serve both the existing uses already present as well as newly proposed uses. Approval of the proposal would allow the City an opportunity to develop the proposed replacement reservoir as a permitted conditional use at a location adjacent to the City's Bolton Pump Station, allowing concurrent use of the existing public facilities that currently serve this site.

The existing Bolton Reservoir and pump station site currently has adequate public services to provide for fire protection, police, emergency medical services, roads, storm water drainage and water service. The proposed reservoir will be an unattended facility with an access driveway and parking area adequate to accommodate service vehicles. The driveway and parking area at the site will be constructed from an access point on Skyline Drive for maintenance vehicle access. As an unattended facility, the proposed reservoir will require minimal traffic for routine maintenance activities. It is anticipated that following construction, the facility may generate one vehicle trip per week on average, similar to the existing reservoir and pump station. The proposed reservoir will have on-site stormwater drainage facilities that will be connected to existing City infrastructure. The proposed on-site stormwater management facilities will not require new off-site infrastructure. The proposed reservoir will use currently available on-site water service for reservoir maintenance purposes only. The reservoir will not generate wastewater and therefore will not require sanitary sewer service. The proposed reservoir will not generate solid waste. No additional public facilities are required to serve the property to accommodate the proposed project. The proposed reservoir will provide water service only to lands inside the City's Urban Growth Boundary and, therefore, will not introduce an urban service outside the Urban Growth

Boundary. The development of the proposed reservoir will not result in pressure for conversion of non-urbanizable lands to more intense uses. Reponses are provided below to document that the proposal is consistent with the applicable policies.

General Goals

Policy 11-1: Establish, as the City's first priority, the maintenance of existing services and infrastructure in all areas within the existing City limits.

The proposed reservoir replacement is consistent with the priority of maintaining existing water service and water system infrastructure. The Bolton reservoir is the hub for water storage in the city's water system. It provides backup service and supply to the adjacent lower pressure zones and provides the preferred path for supplying water to the adjacent upper pressure zones. The configuration of the replacement reservoir will improve service pressures for customers near to and served by the reservoir; improve the filling capacity from the SFWB supply; and increase the available storage that can be pumped from the existing Bolton Pump Station to the upper pressure zones in the event of an emergency.

Policy 11-2: Development shall not be approved unless: a. the proposal has adequate access to the transportation, storm drainage, potable water, and sewer systems; and, b. these infrastructures have adequate capacity to serve the development.

As discussed above the proposed reservoir has adequate access to the transportation, storm drainage and potable water systems and these have adequate capacity to serve the proposed replacement reservoir and associated facilities. The reservoir will not generate wastewater and therefore will not require sanitary sewer service. No additional public facilities are required to serve the property to accommodate the proposed project

Policy 11-4: The City, or entities designated in the future by the City, shall be the primary provider of the following services and facilities: a. Water supply, storage, and distribution

The proposed replacement reservoir at the existing site supports the City's ability to provide water supply, storage and distribution.

Policy 11-5: Where appropriate, monitor, coordinate with, and regulate the activities of the following, as they affect existing and future residents and businesses.
   a. Water supply

The proposed improvements are consistent with the adopted Water System Master Plan. The master plan coordinates existing and future water supply needs with the associated needed water system infrastructure.

Policy 11-10: Assure all visible public facilities are constructed with attractive design and materials where appropriate.

The proposed reservoir will be partially buried, with approximately 5 feet of wall exposure on the south side facing Skyline Drive, increasing to approximately 15 feet of wall exposure on the north side. The exposed concrete reservoir wall will have a topcoat of shotcrete, which has a gray sandy textured finish. When viewed from the site frontage, the top of the reservoir roof will be several feet lower than Skyline Drive. As part of the project, the existing pump station roof which is a flat roof with an industrial appearance, will be replaced with a new pitched standing seem metal roof more in character with the residential neighborhood. Aesthetic design treatments will include proposed vegetative screening and black coated chain link fencing along the back and side property lines, and a black decorative fence and native landscaping along the front of the site. Existing trees along the frontage will be preserved to the fullest extent possible. Proposed site screening and landscaping will include removal of invasive species and restoration and reforestation with drought tolerant native species. Sample photos of the exposed exterior reservoir wall and the Concrete Reservoir Section are included in the Appendices.

Policy 11-11: Assure that costs for new infrastructure and the maintenance of existing infrastructure are borne by the respective users except when it is determined that improvements are of benefit to the whole community, or that a different financing mechanism is more appropriate.

The proposed project benefits the whole community and is primarily a maintenance project. There is some capacity which has been identified for serving future growth within the existing UGB. The funding for this capacity comes from System Development Charges, which will be repaid through future growth.

Policy 11-12: Whenever feasible, utilize environmentally sensitive materials and construction techniques in public facilities and improvements.

The reservoir will be constructed primarily of concrete and reinforcing steel. Other materials are not available in the industry for buried reservoir construction. The applicant has obtained adjacent staging and stockpiling area which will reduce the amount of soil needed to be removed from the site and returned to backfill around the completed structure.

Goal 11, Section 2: Water System

Policy 11-2-1: Establish the City's Water Master Plan, 1999, which is a supporting document of the Comprehensive Plan, as a guide for development of future water storage and distribution facilities. A list of the planned water system projects shall be included in the public facilities plan summary required under Public Facilities and Services General Action Item 1., and;

Policy 11-2-2: Coordinate water service to future users to allow for the most efficient provision of service within the City and projected subsequent expansion of the City limits within the Urban Growth Boundary as it existed in October 2002, calculated to serve a buildout population not to exceed 31,000.

Response to Policy 11-2-1, and 11-2-2: As discussed in the background section above, the proposal of the replacement reservoir at this location to address system wide storage deficits for developable land within the current UGB is a direct fulfilment of an item in the City's Water System Mater Plan (WSMP). The WSMP recommended construction of improvements to address system wide storage deficits within the City. The City opted to proceed with the WSMP system wide storage alternative "Approach B – Storage and Emergency Supply Improvement", which includes the Bolton Reservoir replacement. The City's WSMP states (on pages 6-14, 6-15):

> *"Bolton Reservoir Replacement:* Construction of a new ground level reservoir to replace the existing Bolton Reservoir would address the current issues with the long-term maintenance of the Bolton Reservoir as well as the 0.8 mg deficit in the Willamette pressure zone and the 0.4 mg deficit in the Robinwood pressure zone. The capacity of the Bolton Reservoir Replacement will depend on a number of factors as previously discussed. For the purposes of this analysis, it is recommended that a 4.0 mg reservoir be constructed to replace the existing Bolton Reservoir. This reservoir volume provides replacement capacity for the existing Bolton Reservoir of 2.0 mg, addresses the combined storage deficit of the Willamette and Robinwood pressure zones of 1.2 mg and provides an additional 0.8 mg of storage to offset emergency supply needs."

The proposed 4.0 MG reservoir at the existing Bolton Reservoir site would provide a lower total cost of storage when compared to the alternative of acquiring sites and building individual facilities for each pressure zone. Because of how the water system is operated, the Bolton Reservoir acts as a hub for the other pressure zones. Pressure zones have storage needs that must be met at the pressure zone directly, such as fire suppression and operational needs, as well as an emergency storage component that can be aggregated system-wide. The 4.0 million gallon size will meet the needs of the Bolton pressure zone which the reservoir directly serves, as well as existing storage deficiencies in the Willamette and Robinwood pressure zones, which can also be efficiently served from the proposed reservoir. The proposed reservoir configuration will allow for more usable storage available to pump to the upper pressure zones in the event of an operational disruption or emergency.

The proposed 4.0 MG reservoir is consistent with the WSMP which determined future water supply requirements and recommended improvements that correct existing system deficiencies and provide for future system needs under the approximate twenty (20) year planning period through 2030. The WSMP study area included the City's existing water service area and all areas within the City's existing Urban Growth Boundary (UGB). The City's water service area includes all areas within the City's current City limits, as well a few homes outside the UGB that receive extra-territorial water service from the City. The proposed 4.0 MG reservoir will provide water service only to lands inside the WSMP study area, and will not introduce an urban service to new customers outside the UGB. The development of the proposed reservoir will not result in pressure for conversion of non-urbanizable lands to more intense uses.

Policy 11-2-3: Require funding for the installation of new water storage and distribution facilities to be the responsibility of the property owners/developers or those receiving direct benefit from those facilities. Where appropriate, the City may participate in the development of those facilities to the extent that they benefit residents or businesses in addition to those directly involved, or if they improve the overall efficiency of the system.

The proposed project benefits the whole community and is primarily a maintenance project. There is some capacity which has been identified for serving future growth within the existing UGB. The funding for this capacity comes from System Development Charges, which will be repaid through future growth.

Goal 11, Section 3: Storm Drainage

Policy 11-3-6: Require that construction practices for all land development projects, private and public, be conducted in such a way as to avoid exposing cuts, grading areas, and trenches to stormwater so that soil erosion is minimized, and soil will not be washed into natural drainage areas.

The construction contractor will be required to obtain and comply with an Oregon DEQ National Pollutant Discharge Elimination System (NPDES) Stormwater Discharge Permit #1200-C. As part of the construction contract requirements for the project, the construction contractor will prepare a project specific sedimentation and erosion control plan in accordance with City, County, State and Federal requirements referenced in the construction contract documents. Prior to any ground disturbing activities, temporary sediment and erosion control measures will be established and functional. Typical temporary erosion control measures will include silt fencing and staked hay bales and/or biofilter bags installed along the downslope limits of disturbed areas to contain sediment. The contractor, and the City's construction representatives, will monitor the sedimentation and erosion control measures during construction. Areas of failure will be identified and repaired immediately to prevent potential for downstream sedimentation. All areas disturbed by the construction activity will be revegetated after construction for permanent protection against erosion. Additional stabilization measures including mulching and erosion control matting may be used if determined to be necessary based on site conditions. Temporary sedimentation and erosion control measures will be maintained until permanent vegetation has been established and upslope areas have been permanently stabilized.

Policy 11-3-8: Encourage use of permeable surfaces in developments.

The vehicle access areas will be a permeable gravel surface outside of the paved driveway leading to the pump station. The paved access is required for fire apparatus (truck) access and will provide improved access for infrequent pump station maintenance vehicles and staff vehicle access.

Goal 11, Section 4: Fire and Police

Policy 11-4-1: Ensure that police and fire protection service providers are closely involved with land use decisions that have implications for the provision of emergency services and crime prevention.

The existing Bolton Reservoir and Pump Station site currently has adequate public services to provide for fire protection, police and emergency medical services. The proposed replacement reservoir, being a concrete covered structure filled with water, presents limited fire risk. Adequate emergency vehicle access will provided with the new site improvements. The proposed reservoir will have a concrete roof with locking security hatches, as opposed to the floating rubber cover on the existing tank. This robust structural feature, along with the proposed perimeter fence and motion controlled security lighting will greatly improve security of the City's water supply.

Goal 13: Energy Conservation

The proposed use for a replacement water storage reservoir at this site is consistent with the applicable policies of Goal 13. Reponses are provided below to document that the proposal is consistent with the applicable policies.

Policy 13-6: Encourage the use of energy-conscious design and materials in all public facilities.

The reservoir will be an unattended facility with electrical service to supply site lighting for security, telemetry controls and monitoring equipment, which all have minimal energy consumption. Site lighting will include both automatic motion and photoelectric controls to conserve energy by minimizing usage at night and preventing daytime use. The proposed replacement reservoir constructed at the existing reservoir site would be supplied by the existing SFWB pumps, without the need for additional pumping head and the additional energy consumption that would be required if the reservoir were located at an elevation higher than the existing site's ideal topography.

Sunset Neighborhood Plan

The proposed use for a replacement water storage reservoir at this site is consistent with the Sunset Neighborhood Plan (SNP). Reponses are provided below to document that the proposal is consistent with the applicable policies.

SNP, Goal 3: Ensure that the natural and scenic environment of Sunset Neighborhood is well-maintained and preserved.

SNP Policy 3-1: "Plant new, and protect existing, trees to improve the established tree canopy."

As part of the proposed project, new trees will be planted and existing trees protected to improve the established tree canopy. Existing trees on the site include a dense stand of Bigleaf Maple north of the existing reservoir and a stand of Douglas-fir south of the existing reservoir.

The stand of Bigleaf Maples located along the north property boundary is heavily infested with invasive English ivy, which is growing up the trunks and into the crowns of many trees. While the stand appears in fair condition as an undisturbed intact group at this time, the ivy can be expected to overtop and kill the trees. In addition, many of these trees have multiple leaders, trunks decay, and small high live crowns. Areas northeast of the reservoir will require tree removal where mass excavation and slope grading improvements are required. Some clearing and tree removal will be needed in the utility easements north of the City owned property to complete the proposed piping replacement. Final grading and revegetation will include both screening trees and reforestation with native tree species.

The Douglas-fir stand located along Skyline Drive includes trees of variable condition, but the group as a whole is considered to be in good condition. Many of these trees have old broken tops with new leaders, with some better adapted and more stable than others. Smaller trees that are being dominated by the larger trees which create the canopy cover. The group is undergoing natural stand dynamics and dominant trees are outcompeting and suppressing less vigorous trees. Tight excavation shoring will be used for the proposed reservoir excavation and construction to minimize tree removal to the south and southeast of the proposed reservoir location. The tree stand containing significant trees in the southeast corner of the property will be maintained to the maximum extent possible.

The proposed tree removal includes approximately 81 non-significant trees, mostly Bigleaf Maples, and 28 potentially significant trees, mostly Douglas-fir. The proposed planting includes approximately 80 trees with a mix of Douglas-fir, Vine Maples, Incense Cedar, and Western Red Cedar. Proposed site screening and landscaping will include removal of invasive English ivy and blackberry, removal of unhealthy or undesirable trees as recommended by the project arborist, and restoration with drought tolerant native species. The trees proposed for planting as landscaping and screening along the side and back property lines include Incense Cedar, Douglas-fir and Western Red Cedar. Vine Maples and assorted native shrubs are proposed for planting under the existing Douglas-fir stand to the south and southeast of the reservoir, to replace the non-native grass with an appropriate understory to improve wildlife habitat.

SNP Policy 3-5: "Protect parks from natural and human encroachment."

The proposed reservoir replacement project is consistent with this policy as it addresses several key policy actions including increasing protection of Wilderness Park, and removing invasive species and promoting native species on public properties. The proposed site is an existing City owned site with a municipal water storage reservoir and pump station located across the street from Wilderness Park. Several alternative sites were identified and evaluated as part of the initial preliminary engineering phase of the reservoir replacement project. Through that evaluation only three viable site alternatives remained, with the top choice being the existing site, followed by two other sites located in Wilderness Park. The advantages of the existing site, which was selected as the preferred site, are its large flat area, its current use as a reservoir site, its minimal impact to natural resources, and its close proximity to the Bolton Pump Station and key water infrastructure. Both properties in Wilderness Park are encumbered by deed restrictions which do not permit uses other than those for park purposes. In addition, the West Linn City Charter Chapter XI, Section 46 specifically identifies water reservoirs as a non-authorized use for City owned parks and open spaces, requiring approval by public vote. The natural resources and habitat of Wilderness Park would be protected by constructing the proposed reservoir at the existing reservoir site instead of the Wilderness Park site alternatives.

Proposed site screening and landscaping will include removal of invasive English ivy and blackberry, removal of unhealthy or undesirable trees as recommended by the project arborist, and restoration with drought tolerant native species. The trees proposed for planting as landscaping and screening along the side and back property lines include Incense Cedar, Douglas-fir and Western Red Cedar. Vine Maples and assorted native shrubs are proposed for planting under the existing Douglas-fir stand to the south and southeast of the reservoir, to replace the non-native grass with an appropriate understory to improve wildlife habitat.

60.070(B): An approved conditional use or enlargement or alteration of an existing conditional use shall be subject to the development review provisions set forth in Chapter 55 CDC.

This application includes the Class II Design Review in accordance with CDC Chapter 55.

60.070(C): The Planning Commission may impose conditions on its approval of a conditional use which it finds are necessary to assure the use is compatible with other uses in the vicinity.

The site has been a reservoir site and served as the hub of the City's water system since 1915 when the existing reservoir was constructed. The proposed use is not changing from its current use. The site currently contains the existing 2.5 MG Bolton Reservoir, existing Bolton Pump Station, Old Bolton Pump Station and existing storage building. The proposed reservoir structure will occupy an area of approximately 0.54 acres, which is slightly smaller than the 0.64 acre footprint of the existing reservoir. The existing reservoir, old pump station and storage building will be demolished and removed as part of the proposed reservoir project. The site is of adequate size to accommodate the replacement reservoir structure and associated facilities, including the stormwater detention pond and water quality swale, while leaving adequate space for a future replacement building and maintenance vehicle access to the proposed improvements and existing pump station. As an unattended facility, the proposed reservoir will require minimal traffic for routine maintenance activities. It is anticipated that following construction, the facility may generate one vehicle trip per week on average, similar to the existing reservoir and pump station.

60.070(D): Aggregate extraction uses shall also be subject to the provisions of ORS 541.605.

These provisions do not apply as this project is not an aggregate extraction use.

60.070(E): The Historic Review Board shall review an application for a conditional use, or to enlarge a conditional use on a property designated as a historic resource, based on findings of fact that the use will:

1. Preserve or improve a historic resource which would probably not be preserved or improved otherwise; and
2. Utilize existing structures rather than new structures.

This project is not subject these criteria as this project was determined to be not eligible for the National Register per the State Historical Preservation Office (SHPO) in consultation with the City of West Linn Planning Department. The letters of finding from the SHPO are included in the Appendices.

60.080: Site Plan and Map

A Site Plan (Figure 1) was prepared in accordance with the requirements of CDC Chapter 55 and 60 and is included in the Appendices.

60.090 Additional Criteria for Transportation Facilities (Type II)

The proposed use is not a transportation facility, therefore CDC 60.090 is not applicable.

60.100 Additional Criteria for Schools and Other Government Facilities

The proposed use is not a school or other government facility that attracts a regular and significant volume of users, therefore CDC 60.100 is not applicable.

IV. CLASS I & II DESIGN REVIEW NARRATIVE

The following are the Applicant's responses to the approval criteria of CDC Chapter 55. Several figures and maps required per Section 55.070 are attached and should be reviewed with this narrative. The reservoir replacement is a Class II Design Review and all portions of the narrative apply.

The proposed roof construction requires a Class I Design Review per CDC 55.020(A)(5): "Minor modifications and/or upgrades of pump stations, reservoirs, and storm detention facilities." CDC 55.100(B) (1) through (6) apply to the roof elements.

55.070 SUBMITTAL REQUIREMENTS
55.070(A): The design review application shall be initiated by the property owner or the owner's agent, or condemnor.

This application was initiated by the City of West Linn as the property owner.

55.070(B): A pre-application conference, per CDC 99.030(B), shall be a prerequisite to the filing of an application.

A pre-application conference was conducted on February 5, 2015. The conference summary notes are included in the Appendices.

55.070(C): Documentation of any required meeting with the respective City-recognized neighborhood association per CDC 99.038.
The pre-application conference summary notes identified the neighborhood associations of Sunset, Bolton and Hidden Springs/Rosemont Summit for required neighborhood contact meetings under CDC 99.038. A single meeting was conducted for all the applicable neighborhood associations on Thursday April 23, 2015, at 7:00 p.m. in the West Linn Public Library Community Room. To demonstrate compliance with CDC 99.038 the following documentation has been included with the application in the Appendices:

1. A copy of the certified letter to the neighborhood association with a copy of return receipt;
2. A copy of the letter to officers of the association and to property owners within 500 feet, including an affidavit of mailing and a copy of the mailing list containing the names and addresses of such owners and residents;
3. A copy of the required posted notice, along with an affidavit of posting;
4. A copy of the minutes of the meeting;
5. An audiotape of the meeting

55.070(D): The applicant shall submit a completed application form and:
55.070(D)(2)(a):a. A site analysis (CDC 55.110);

The Site Analysis is included in the Appendices.

55.070(D)(2)(b): A site plan (CDC 55.120);

The Site Plan (Figure 1) is included in the Appendices.

55.070(D)(2)(c): A grading plan (CDC 55.130);

The Grading Plan is included in the Appendices.

55.070(D)(2)(d): Architectural drawings, indicating floor plan and elevation (CDC 55.140);

The Concrete Reservoir Section (Figure 4) and Proposed Reservoir Floor and Roof Plan (Figure 10) are included in the Appendices.

The pump station Architectural Elevations (Figure 4) showing the proposed roof replacement, and the existing building floor plan (Figure 9) are included in the Appendices.

55.070(D)(2)(e): A landscape plan (CDC 55.150);

The Landscape Plan is included in the Appendices.

55.070(D)(2)(f): A utility plan appropriate to respond to the approval criteria of CDC 55.100(I)(1) through (5) relating to streets, drainage, municipal water, sanitary sewers, solid waste, and recycling storage;

A utility plan is not applicable for the replacement of the reservoir. The existing and proposed facilities do not require water, sewer, solid waste or recycling service or new streets. The site storm drainage will not increase peak runoff per City Standards, and will not require off-site improvements beyond what is shown in the utility easement on the Site Plan. The proposed stormwater management is illustrated on the Site Plan (Figure 1).

55.070(D)(2)(g): A light coverage plan with photometric data, including the location and type of outdoor lighting, with specific consideration given to compliance with CDC 55.100(J) pertaining to crime prevention and, if applicable, CDC 46.150(A)(13) pertaining to parking lot lighting;

The Light Coverage Plan is included in the Appendices.

55.070(D)(2)(h): If staff determines before or during the pre-application conference that the land use is expected to generate noise that may exceed DEQ standards, the application shall include a noise study conducted by a licensed acoustical engineer that demonstrates that the application and associated noise sources will meet DEQ standards. Typical noise sources of concern include, but are not limited to, vehicle drive-throughs, parking lots, HVAC units, and public address systems.

Staff has not made a determination that a noise study is required, therefore this criteria is not applicable.

55.070(D)(2)(i): Documents as required per the Tree Technical Manual.

The Arborist Report is included in the Appendices.

55.070(D)(3): A narrative, based on the standards contained in this code, which supports any requested exceptions as provided under CDC 55.170.

No exceptions are requested.

55.070(D)(4): Submit full written responses to approval criteria of CDC 55.100 for Class II design review, or CDC 55.090 for Class I design review, plus all applicable referenced approval criteria.

Full written responses to approval criteria of CDC 55.100 for Class II design review is included as Section III - Class II Design Review Narrative.

55.070(E): The applicant shall submit samples of all exterior building materials and colors in the case of new buildings or building remodeling.

The exposed concrete reservoir wall will have a topcoat of shotcrete, which has a gray sandy textured finish. The pump station standing seam metal roof will be forest green. The gable-end siding will be light brown or tan colored clapboard siding. A photo of shotcrete covered exterior reservoir wall are included in the Appendices. Along with the architectural drawings and sections in the Appendices, this information provides a good understanding of the proposed building materials. A waiver is requested for the exterior building material samples submittal requirement.

55.070(F): The applicant shall pay the required deposit and fee. (Ord. 1401, 1997; Ord. 1408, 1998; Ord. 1442, 1999; Ord. 1613 § 11, 2013; Ord. 1621 § 25, 2014; Ord. 1622 § 14, 2014)

Municipal projects have no applicable fees.

55.100 APPROVAL STANDARDS – CLASS II DESIGN REVIEW
55.100: The approval authority shall make findings with respect to the following criteria when approving, approving with conditions, or denying a Class II design review application.
55.100(A): The provisions of the following chapters shall be met:
55.100(A)(1): Chapter 34 CDC, Accessory Structures, Accessory Dwelling Units, and Accessory Uses.

As a future project, it is proposed to construct a storage building as a replacement for the existing storage building to be removed as part of the reservoir replacement. This structure will be an Accessory Structure as defined by CDC 34.020.

As shown on the Site Plan in the Appendices, the future building will be able to meet the required setbacks of CDC 11.070(5) as shown below.

Setback Description	Required Minimum	Proposed Setback
Front	20 feet	40 feet
Side	7.5 feet	15 feet
Rear	20 feet	300 feet

The proposed future building is anticipated to be a wood-framed single story storage building with an appearance similar to existing neighborhood residential properties.

55.100(A)(2): Chapter 38 CDC, Additional Yard Area Required; Exceptions to Yard Requirements; Storage in Yards; Projections into Yards.

The proposed reservoir structure and existing pump station structure will both be more than three feet from the property line, more than 25 feet from the nearest street, will not have any projections extending into the front or rear yard (such as porches, decks or balconies) by more than five feet. The front yard of the property will not be used for storage of vehicles or trailers.

55.100(A)(3): Chapter 40 CDC, Building Height Limitations, Exceptions.

Repealed by Ordinance 1604.

55.100(A)(4): Chapter 42 CDC, Clear Vision Areas.

These provisions are met. The clear vision area for the 24-foot wide driveway as computed per CDC 42.040 are met. The clear vision area is illustrated on the Site Plan (Figure 1) in the Appendices.

55.100(A)(5): Chapter 44 CDC, Fences.

These provisions are met.

A new security fence six feet in height will be installed along the side and back yard property lines within the required side and back yards. The fence will be a black coated chain link fence that will not obscure vision.

A decorative metal security fence six feet in height will also be installed along the front of the property outside of the front yard setback distance. The fence will meet the clear vision requirements of CDC 42.

55.100(A)(6): Chapter 46 CDC, Off-Street Parking, Loading and Reservoir Areas.

a. **46.020:** Parking for employee access will continue to use the existing paved area in front of the pump station.

b. **<u>46.090:</u>** Space for parking two vehicles on the paved area in front of the pump station currently exists. There is ample paved and gravel areas to accommodate additional parking if needed.

c. **<u>46.130:</u>** The proposed use does not receive or distribute material or merchandise requiring off-street loading spaces.

d. No parking will be provided for the public.
e. No bicycle facilities will be placed on this site.

55.100(A)(7): Chapter 48 CDC, Access, Egress and Circulation.

a. **<u>48.040:</u>** The service drive will be a 24-foot width accommodating two-way traffic.

b. **<u>48.060:</u>** The access curb cut will be more than 150 feet from any intersection and more than 150 feet from the adjacent property driveway.

55.100(A)(8): Chapter 52 CDC, Signs.

There will be no signs on the site with the proposed use, therefore Chapter 52 is not applicable.

55.100(A)(9): Chapter 54 CDC, Landscaping.

a. **<u>54.020(D):</u>** There are no heritage trees identified on the project site.

b. **<u>54.020(E)(3):</u>** More than 20 percent of the site will be landscaped. All exposed ground not otherwise landscaped will be covered in drought resistant grass.

c. **<u>54.020(G): Water Resource Area (WRA):</u>**
The project site is not in a WRA.

d. **<u>54.050: Installation:</u>**
All landscaping to be added to the site will comply with the requirements of installation as laid out in this section.

55.100(B): Relationship to the natural and physical environment.
55.100(B)(1):

No heritage trees are identified on the project site.

55.100(B)(2)(b): Non-residential and residential projects on non-Type I and II lands shall set aside up to 20 percent of the area to protect trees and tree clusters that are determined to be significant, plus any heritage trees.

The 3.23 acres site includes approximately 2.81 acres of non-Type I and Type II lands (as defined in Chapter 2 of the CDC as having more than 25 percent slope). There are approximately 0.78 acres of significant trees on the existing site. 0.58 acres of the significant tree area (74 percent) will be preserved. Approximately 20.6 percent of the non-Type I and Type II land area will be used for significant tree preservation. This condition is met.

The significant tree cluster to be preserved is discussed in the Arborist Report and illustrated within the tree protection fencing shown on the Erosion Control Measures (Figure 3) in the Appendices.

55.100(B)(3): The topography and natural drainage shall be preserved to the greatest degree possible.

The site topography will continue to slope northward and match elevations along the property boundary without use of retaining walls. Uncollected stormwater runoff will continue to travel overland. Collected stormwater will be continue to be conveyed by pipe to Bolton Creek to the northwest, after detention and water quality treatment. Existing overland runoff is naturally conveyed to Bolton Creek by a combination of the stormwater system on Caufield Street and groundwater infiltration.

55.100(B)(4): The structures shall not be located in areas subject to slumping and sliding. The Comprehensive Plan Background Report's Hazard Map, or updated material as available and as deemed acceptable by the Planning Director, shall be the basis for preliminary determination.

The Natural Hazard Mitigation Plan, Map 16 Potential Landslides included in the Appendices identifies "DOGAMI Potential Landslides" and areas with slopes that exceed 25 percent. The proposed reservoir site is not within the areas identified as DOGAMI Potential Landslides on Map 16, but it does have slopes within the property limits that exceed 25 percent as shown the attached Site Analysis Map.

The Natural Hazard Mitigation Plan, Map 17 Landslide Vulnerability Analysis included in the Appendices, identifies all areas with slopes that exceed 25 percent as "Landslide Hazard Areas". The existing reservoir is identified as one of the "Assets and Infrastructure within Landslide Area" on Map 17. The attached Site Analysis Map shows areas within the site with slopes that exceed 25 percent. As part of recent preliminary engineering, geotechnical investigation and analysis was performed which confirmed the geotechnical suitability of the site, with recommendations to address potential hazards related to steep slopes identified on Map 17. As part of the investigation a small shallow flow slide was reported to have occurred in the 1970's near the northeast corner of the Bolton Reservoir site, with likely causes related to a water main break and fill that been placed at the top of the slope during the original reservoir construction. The new reservoir as planned, will not adversely affect the existing slope stability. The planned ground improvement beneath the reservoir, removal of soil at the top of the slope along the north side of the site, and the gravel pad and subdrainage system around and beneath the reservoir will improve local factors of safety as they relate to potential reservoir instability. The proposed reservoir replacement will move the new structure away from the steep slopes and further reduce the risk of landslides associated with the steep slope.

As part of the geotechnical investigation, more recent updated mapping available from DOGAMI, SLIDO (Statewide Landslide Information Database of Oregon, Release 2, 2011) was reviewed. The existing reservoir and proposed project site were reported to be within a very large ancient landslide mapped by DOGAMI. This landslide is a prehistoric, deep-seated translational rock landslide referred to as Canby 133 that covers approximately 170 acres of West Linn's northern slopes. Based on the investigation it was reported that the Canby 133 landslide is likely on the order of at least 15,000 to 20,000 years old, and the risk of significant movement of the large ancient landslide within the design life of the reservoir is expected to be low. A detailed discussion of landslide hazards based on the recent geotechnical investigation and analysis, and site-specific seismic hazard study is included in the response for Goal 7.

55.100(B)(5): There shall be adequate distance between on-site buildings and on-site and off-site buildings on adjoining properties to provide for adequate light and air circulation and for fire protection.

The proposed reservoir is located a minimum of 50 feet from any property line. The reservoir is constructed of concrete and metal and filled with water and presents a negligible risk for causing or being significantly damaged by a fire.

55.100(B)(6): Architecture.

The proposed reservoir design is consistent with the current structures on the site. The proposed pump station roof replacement, which will have a gabled-end design, will better match the residential nature of the structures on the adjacent properties.

55.100(B)(7): Transportation Planning Rule (TPR) compliance.

The Transportation Planning Rule does not apply to major utility uses. This project site will be closed to the public.

55.100(C): Compatibility between adjoining uses, buffering, and screening.

Landscape screening will be installed along the side and rear yard. Rooftop air cooling and heating systems and other mechanical equipment are not elements of the reservoir structure.

55.100(D): Privacy and noise.

The reservoir operation does not produce noise. Site lighting will be provided for site security and theft and vandalism deterrence meeting City standards. The directed lighting will be motion activated and photoelectric controlled. See the attached Light Coverage Plan in the Appendices.

55.100(E): Private outdoor area. This section only applies to multi-family projects.
55.100(F): Shared outdoor recreation areas. This section only applies to multi-family projects and projects with 10 or more duplexes or single-family attached dwellings on lots under 4,000 square feet
55.100(G): Demarcation of public, semi-public, and private spaces. The structures and site improvements shall be designed so that public areas such as streets or public gathering places, semi-public areas, and private outdoor areas are clearly defined in order to establish persons having a right to be in the space, to provide for crime prevention, and to establish maintenance responsibility.

This site does not have any public spaces outside of the roadway right-of-way. The site will be fenced to reduce crime.

55.100(H): Public transit.

There will not be a need for public transportation for this facility.

55.100(I): Public facilities. An application may only be approved if adequate public facilities will be available to provide service to the property prior to occupancy.

1. Streets: No new streets will be added.

2. Municipal water: Fire suppression capacity will be provided from the fire hydrant on Skyline Drive which is supplied by gravity from the Horton Reservoir. The proposed reservoir is less than 200 feet from the fire hydrant.

3. Sanitary sewers: The proposed reservoir does not require a sanitary sewer connection.

4. Solid waste and recycling storage areas: There will not be any solid waste or recycling waste storage areas for operation of the proposed reservoir.

55.100(J): Crime prevention and safety/defensible space.

1. Windows, service areas, mailboxes, and waste facilities are not included in this project.

2. Refer to the lighting plan. The lighting will cover the steeper slopes along the access routes and illuminate the crime or vandalism prone areas which include the top of the proposed reservoir, the pump station entry doors and the standby power generator. Lighting will be downward directed with motion sensors.

3. **Lines of sight shall be reasonably established so that the development site is visible to police and residents.** Vegetative screening will allow activity on the site to be detected by adjacent neighbor. The site will be visible from the frontage road by police, city staff and the general public.

55.100(K): Provisions for persons with disabilities.

The site use is Major Utility and the site is closed to the public, and therefore these provisions are not applicable.

55.100(L): Signs.

There will be no signs on the site with the proposed use.

55.100(M): Utilities.

All affected and new utilities on the site will be located underground.

55.100(N): Wireless communication facilities (WCFs). (This section only applicable to WCFs.)

No wireless communications facilities are included in this project, therefore this criteria is not applicable.

55.100(O): Refuse and recycling standards.

Operation of the proposed reservoir and pump station site does not generate any solid waste or recyclable waste, therefore this criteria is not applicable. Waste storage areas are not required.

55.110 SITE ANALYSIS
55.110(A): A vicinity map showing the location of the property in relation to adjacent properties, roads, pedestrian and bike ways, transit stops and utility access.

Refer to the Site Analysis (Figure 2) drawing.

55.110(B): A site analysis on a drawing at a suitable scale (in order of preference, one inch equals 10 feet to one inch equals 30 feet) which shows:
55.110(B)(1): The property boundaries, dimensions, and gross area.

Refer to the Site Analysis (Figure 2) drawing.

55.110(B)(2): Contour lines at the following minimum intervals:

a. Two-foot intervals for slopes from zero to 25 percent; and
b. Five- or 10-foot intervals for slopes in excess of 25 percent.

Refer to the Site Analysis (Figure 2) drawing.

55.110(B)(3): A slope analysis which identifies portions of the site according to the slope ranges as follows:

a. Type I (under 15 percent);
b. Type II (between 15 to 25 percent);
c. Type III (between 25 to 35 percent);
d. Type IV (over 35 percent).

Refer to the Site Analysis (Figure 2) drawing.

55.110(B)(4): The location and width of adjoining streets.

Refer to the Site Analysis (Figure 2) drawing.

55.110(B)(5): The drainage patterns and drainage courses on the site and on adjacent lands.

Refer to the Site Analysis (Figure 2) drawing.

55.110(B)(6): Potential natural hazard areas including:

a. Floodplain areas pursuant to the site's applicable FEMA Flood Map panel:

The project site is outside of the flood plain, therefore this criteria is not applicable.

b. Water resource areas as defined by Chapter 32 CDC:

The project site is not identified in the City's Water Resource Area Map, therefore this criteria is not applicable..

c. Landslide areas designated by the Natural Hazard Mitigation Plan, Map 16

The Natural Hazard Mitigation Plan, Map 16 Potential Landslides included in the Appendices identifies DOGAMI Potential Landslides and areas with slopes that exceed 25 percent. The proposed reservoir site is not within the areas identified as DOGAMI Potential Landslides on Map 16, but it does have slopes within the property limits that exceed 25 percent as shown the attached Site Analysis Map. A detailed discussion of landslide hazards based on subsequent geotechnical investigation and analysis, and site-specific seismic hazard study is included in the response for Goal 7.

d. Landslide vulnerable analysis areas, designated by the Natural Hazard Mitigation Plan, Map 17.

The Natural Hazard Mitigation Plan, Map 17 Landslide Vulnerability Analysis included in the Appendices, identifies all areas with slopes that exceed 25 percent as “Landslide Hazard Areas”. The existing reservoir is identified as one of the “Assets and Infrastructure within Landslide Area” on Map 17. The attached Site Analysis Map shows areas within the site with slopes that exceed 25 percent. A detailed discussion of landslide hazards based on subsequent geotechnical investigation and analysis, and site-specific seismic hazard study is included in the response for Goal 7.

55.110(B)(7): Resource areas including:

a. Wetlands;
b. Riparian corridors;
c. Streams, including intermittent and ephemeral streams;
d. Habitat conservation areas; and
e. Large rock outcroppings.

The City’s Water Resource Area Map does not identify any resources on the project site. Adjacent properties include two drainages or creeks. Bolton Creek to the northwest and the unnamed ephemeral drainage to the north are shown on the Site Analysis (Figure 2) drawing. Large rock outcroppings are not present on the site.

55.110(B)(8): Potential historic landmarks and registered archaeological sites. The existence of such sites on the property shall be verified from records maintained by the Community Development Department and other recognized sources.

The project site is not identified by the Community Development Department as having the potential for historic landmarks. The State Historic Preservation Office did not identify any archaeological or historical interests at the project site. Indian cairns, graves and other significant archeological resources uncovered during construction or excavation will be preserved intact until a plan for their excavation or reinterment can been developed by the State Historic Preservation Office.

55.110(B)(9): Identification information including the name and address of the owner, developer, project designer, lineal scale and north arrow.

Refer to the Site Analysis (Figure 2) drawing.

55.110(B)(10): Identify Type I and II lands in map form. Provide a table which identifies square footage of Type I and II lands also as percentage of total site square footage.

Refer to the Site Analysis (Figure 2) drawing.

55.120 SITE PLAN

55.120: The site plan shall be at the same scale as the site analysis (CDC 55.110) and shall show:
55.120(A): The applicant's entire property and the surrounding property to a distance sufficient to determine the relationship between the applicant's property and proposed development and adjacent property and development.

Refer to the Site Plan (Figure 1) drawing.

55.120(B): Boundary lines and dimensions for the perimeter of the property and the dimensions for all proposed lot or parcel lines.

Refer to the Site Plan (Figure 1) drawing.

55.120(C): Streams and stream corridors.

Refer to the Site Plan (Figure 1) drawing. The Bolton Creek is located to the northwest of the project site.

55.120(D): Identification information, including the name and address of the owner, developer, project designer, lineal scale and north arrow.

Refer to the Site Plan (Figure 1) drawing.

55.120(E): The location, dimensions, and names of all existing and proposed streets, public pathways, easements on adjacent properties and on the site, and all associated rights-of-way.

Refer to the Site Plan (Figure 1) drawing.

55.120(F): The location, dimensions and setback distances of all:
55.120(F)(1): Existing and proposed structures, improvements, and utility facilities on site;

Refer to the Site Plan (Figure 1) drawing.

55.120(F)(2): Existing structures and driveways on adjoining properties.
Refer to the Site Plan (Figure 1) drawing.

55.120(G): The location and dimensions of:

<u>55.120(G)(1): The entrances and exits to the site:</u>
Refer to the Site Plan (Figure 1) drawing.

<u>55.120(G)(2): The parking and circulation areas:</u>
Refer to the Site Plan (Figure 1) drawing.

<u>55.120(G)(3): Areas for waste disposal, recycling, loading, and delivery:</u>
These area are not required for this site, therefore this criteria is not applicable.

<u>55.120(G)(4): Pedestrian and bicycle routes, including designated routes, through parking lots and to adjacent rights-of-way:</u>
There are no on-site pedestrian or bicycle routes on this site, which is closed to the public, therefore this criteria is not applicable.

<u>55.120(G)(5): On-site outdoor recreation spaces and common areas:</u>
These areas are not required for this site, therefore this criteria is not applicable.

<u>55.120(G)(6): All utilities, including stormwater detention and treatment</u>:
Refer to the Site Plan (Figure 1) drawing.

<u>55.120(G)(7): Sign locations:</u>
There will be no signs on the site with the proposed use.

55.120(H): The location of areas to be landscaped.

Refer to the Landscape Plan (Figure 7) drawing.

55.125 TRANSPORTATION ANALYSIS
Certain development proposals required that a Traffic Impact Analysis (TIA) be provided which may result in modifications to the site plan or conditions of approval to address or minimize any adverse impacts created by the proposal. The purpose, applicability and standards of this analysis are found in CDC <u>85.170</u>(B)(2).

The proposed re-development will not affect the number of vehicle trips, internal traffic patterns or site access safety that would require a TIA.

55.130 GRADING PLAN
55.130(A): The location and extent to which grading will take place indicating general contour lines, slope ratios, slope stabilization proposals, and location and height of retaining walls, if proposed.

Refer to the Grading Plan (Figure 5) included in the Appendices.

55.130(B): A registered civil engineer shall prepare a plan and statement that shall be supported by factual data that clearly shows that there will be no adverse impacts from increased intensity of runoff off site, or the plan and statement shall identify all off-site impacts and measures to mitigate those impacts. The plan and statement shall, at a minimum, determine the off-site impacts from a 10-year storm.

The stormwater management plan was developed per City standards. Refer to the Preliminary Stormwater Management Plan in the Appendices.

55.130(C): Storm detention and treatment plans may be required.

The stormwater management plan includes on-site runoff detention to limit peak runoff to the pre-development rate and on-site water quality to treat the 2-year design storm for all impervious areas and other areas which are collected and conveyed to the municipal stormwater system. The existing pump station and paved area runoff will not be changed.

55.130(D): Identification, information, including the name and address of the owner, developer, project designer, and the project engineer.

This information is included in the Grading Plan (Figure 5) in the Appendices.

55.140 ARCHITECTURAL DRAWINGS
Architectural drawings shall be submitted showing:

55.140(A): Building elevations and sections tied to curb elevation;

The existing pump station Architectural Elevations (Figure 4) and proposed Concrete Reservoir Section (Figure 6) are included in the Appendices.

55.140(B): Building materials: color and type; and

The exposed concrete reservoir wall and roof will be concrete. The reservoir roof will have a steel trowel finish. The reservoir walls will have a finish coat of shotcrete, which has a gray sandy textured finish.

The new pump station roof will be a standing seam metal roof with a factory coating. The coating will be Kynar ™ or similar, which is a factory-applied, oven-baked finish based on a polyvinylidene fluoride resin. The proposed roofing color is forest green. The proposed gable-end siding is light brown or tan to complement the existing pump station's exposed concrete walls, which are a light gray color.

A sample photos of the reservoir shotcrete exterior finish is included in the Appendices.

55.140(C): The name of the architect or designer.

The project structural engineer is Peterson Structural Engineers, Inc.
Murray, Smith & Associates, Inc. is the designer for architectural elements.

55.150 LANDSCAPE PLAN
This section does not apply to detached single-family residential subdivisions or partitions, or up to two duplexes or single-family attached dwellings.
55.150(A): The landscape plan shall be prepared and shall show the following:
55.150(A)(1): Preliminary underground irrigation system, if proposed;

The proposed landscaping will not use a permanent irrigation system.

55.150(A)(2): The location and height of fences and other buffering of screening materials, if proposed;

The site fencing is shown on the Site Plan (Figure 1). The vegetative screening is shown on the Landscape Plan.

55.150(A)(3): The location of terraces, decks, patios, shelters, and play areas, if proposed;

Terraces, decks, patios, shelters, and play areas are not proposed.

55.150(A)(4): The location, size, and species of the existing and proposed plant materials, if proposed; and

Refer to the Landscape Plan (Figure 7) in the Appendices.

55.150(A)(5): Building and pavement outlines.

Refer to the Landscape Plan (Figure 7) in the Appendices.

55.150(B): The landscape plan shall be accompanied by:
55.150(B)(1): The erosion controls that will be used, if necessary;

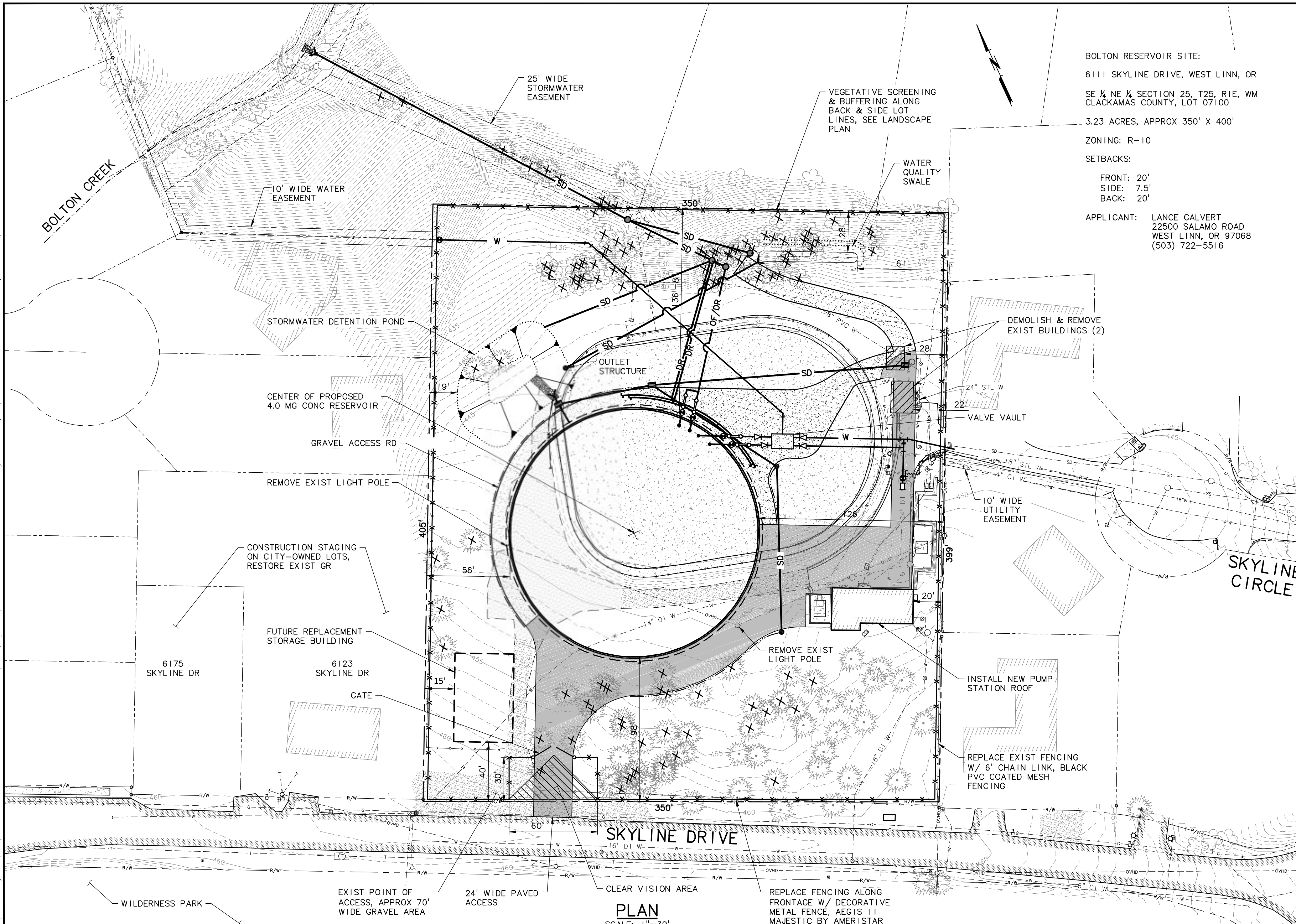
Refer to the Erosion Control Measures in the Appendices for the preliminary erosion control measures. An Oregon DEQ 1200-C Permit will be prepared and obtained by the construction contractor.

55.150(B)(2): Planting list; and

Refer to the Landscape Plan (Figure 7) in the Appendices.

55.150(B)(3): Supplemental information as required by the Planning Director or City Arborist.

Supplemental information has not be requested by the Planning Director or City Arborist.



DESIGN REVIEW AND CONDITIONAL USE PERMIT, PA-15-05

G:\PDX_Projects\14\1586 - Bolton Reservoir Replacement\CAD\Figures\14-1586-201-OR-CUP FIGS.dwg FIG-1 5/21/2015 1:50 PM DKT 20.0s (LMS Tech)

NO.	DATE	REVISION	BY

SCALE: VERT: AS SHOWN; HORIZ: AS SHOWN

NOTICE: IF THIS BAR DOES NOT MEASURE 1" THEN DRAWING IS NOT TO SCALE

PRELIMINARY ONLY — DO NOT USE FOR CONSTRUCTION

MAY 2015

MURRAY, SMITH & ASSOCIATES, INC. Engineers/Planners

DESIGNED: MLM
DRAWN: DKH
CHECKED: MLM
APPROVED: TPB

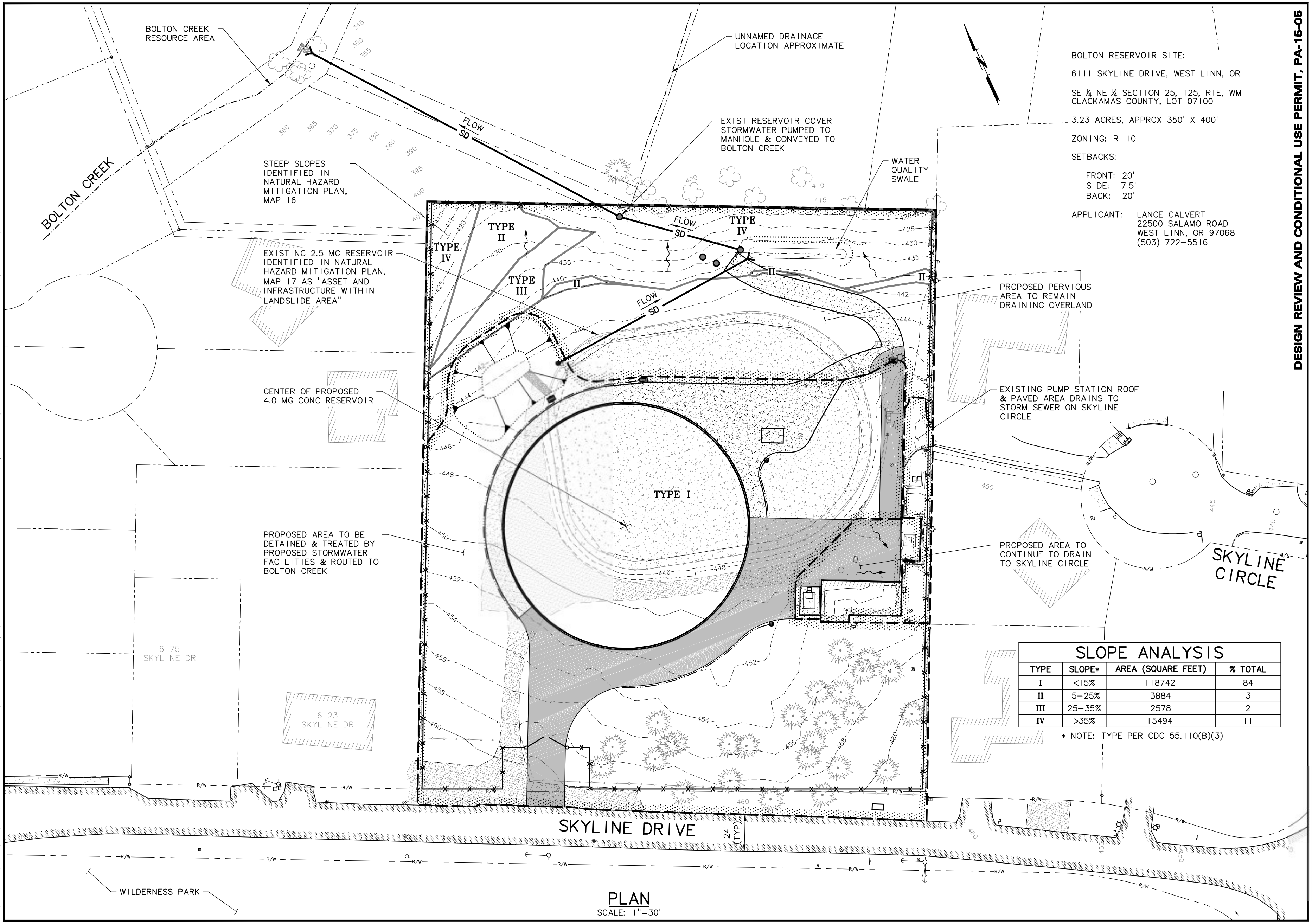
PROJECT NAME: CITY OF WEST LINN, OREGON — BOLTON RESERVOIR REPLACEMENT — PROJECT NO. PW 14-06

SHEET TITLE: SITE PLAN

SHEET FIG-1 of XXX

MSA — Murray, Smith & Associates, Inc. Engineers/Planners
121 S.W. Salmon, Suite 900, Portland, Oregon 97204
PHONE 503-225-9010 FAX 503-225-9022

MSA PROJECT: 14-1586
DATE: MAY 2015



SLOPE ANALYSIS

TYPE	SLOPE*	AREA (SQUARE FEET)	% TOTAL
I	<15%	118742	84
II	15–25%	3884	3
III	25–35%	2578	2
IV	>35%	15494	11

* NOTE: TYPE PER CDC 55.110(B)(3)

PLAN
SCALE: 1"=30'

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DESIGNED: MLM
DRAWN: DKH
CHECKED: MLM
APPROVED: TPB

SHEET FIG-2 of XXX

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PROJECT NAME: CITY OF WEST LINN, OREGON
BOLTON RESERVOIR REPLACEMENT
PROJECT NO. PW 14-06

SHEET TITLE: SITE ANALYSIS

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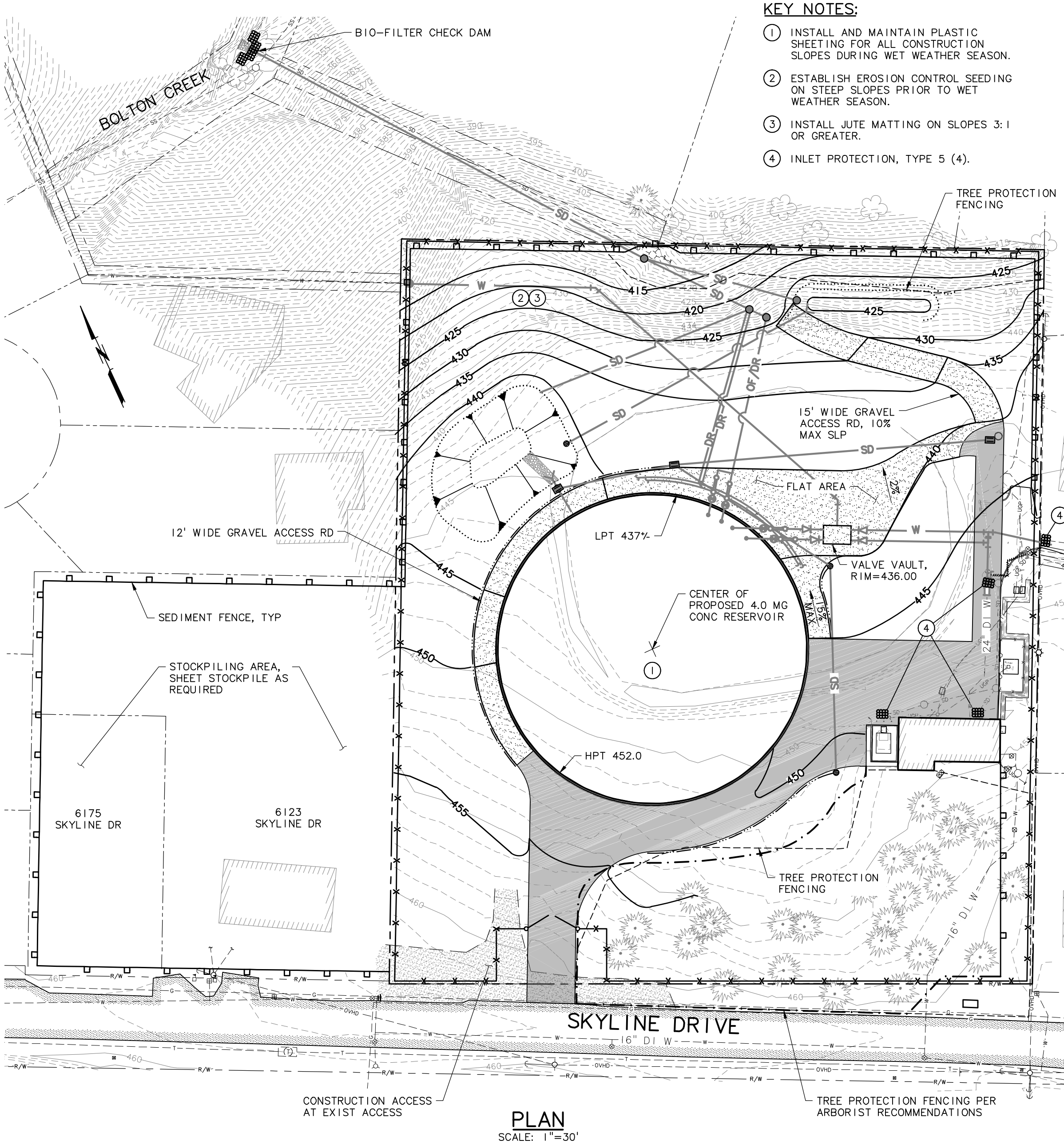
MSA PROJECT: 14-1586.
DATE: MAY 2015

G:\PDX_Projects\14\1586 - Bolton Reservoir Replacement\CAD\Figures\14-1586-201-OR-CUP FIGS.dwg FIG-2 5/21/2015 1:50 PM DKT 20.0s (LMS Tech)



PLAN
SCALE: 1"=30'

KEY NOTES:

1. INSTALL AND MAINTAIN PLASTIC SHEETING FOR ALL CONSTRUCTION SLOPES DURING WET WEATHER SEASON.
2. ESTABLISH EROSION CONTROL SEEDING ON STEEP SLOPES PRIOR TO WET WEATHER SEASON.
3. INSTALL JUTE MATTING ON SLOPES 3:1 OR GREATER.
4. INLET PROTECTION, TYPE 5 (4).

STANDARD EROSION AND SEDIMENT CONTROL PLAN DRAWING NOTES

1. WHEN RAINFALL AND RUNOFF OCCURS DAILY INSPECTIONS OF THE EROSION AND SEDIMENT CONTROLS AND DISCHARGE OUTFALLS MUST BE PROVIDED BY SOME ONE KNOWLEDGEABLE AND EXPERIENCED IN THE PRINCIPLES, PRACTICES, INSTALLATION, AND MAINTENANCE OF EROSION AND SEDIMENT CONTROLS WHO WORKS FOR THE PERMITTEE.

2. CONSTRUCTION ACTIVITIES MUST AVOID OR MINIMIZE EXCAVATION AND CREATION OF BARE GROUND FROM OCTOBER 1 THROUGH MAY 31 EACH YEAR.

3. DURING WET WEATHER PERIOD, TEMPORARY STABILIZATION OF THE SITE MUST OCCUR AT THE END OF EACH WORK DAY.

4. SEDIMENT CONTROLS MUST BE INSTALLED AND MAINTAINED ON ALL DOWN GRADIENT SIDES OF THE CONSTRUCTION SITE AT ALL TIMES DURING CONSTRUCTION. THEY MUST REMAIN IN PLACE UNTIL PERMANENT VEGETATION OR OTHER PERMANENT COVERING OF EXPOSED SOIL IS ESTABLISHED.

5. ALL ACTIVE INLETS MUST HAVE SEDIMENT CONTROLS INSTALLED AND MAINTAINED AT ALL TIMES DURING CONSTRUCTION. UNLESS OTHERWISE APPROVED, A SURFACE MOUNTED AND ATTACHABLE, U-SHAPED FILTER BAG IS REQUIRED FOR ALL CURB INLET CATCH BASINS.

6. SIGNIFICANT AMOUNTS OF SEDIMENT WHICH LEAVES THE SITE MUST BE CLEANED UP WITHIN 24 HOURS AND PLACED BACK ON THE SITE AND STABILIZED OR PROPERLY DISPOSED. THE CAUSE OF THE SEDIMENT RELEASE MUST BE FOUND AND PREVENTED FROM CAUSING A RECURRENCE OF THE DISCHARGE WITHIN THE SAME 24 HOURS. ANY IN-STREAM CLEAN UP OF SEDIMENT SHALL BE PREFORMED ACCORDING TO THE OREGON DEPARTMENT OF STATE LANDS REQUIRED TIME FRAME.

7. SEDIMENT MUST NOT BE INTENTIONALLY WASHED INTO STORM SEWERS, DRAINAGE WAYS, OR WATER BODIES.

8. SEDIMENT MUST BE REMOVED FROM BEHIND ALL SEDIMENT CONTROL MEASURES WHEN IT HAS REACHED A HEIGHT OF 1/3RD THE BARRIER HEIGHT, AND PRIOR TO THE CONTROL MEASURES REMOVAL.

9. CLEANING OF ALL STRUCTURES WITH SUMPS MUST OCCUR WHEN THE SEDIMENT RETENTION CAPACITY HAS BEEN REDUCED BY 50% AND AT COMPLETION OF PROJECT.

10. ANY USE OF TOXIC OR OTHER HAZARDOUS MATERIALS MUST INCLUDE PROPER STORAGE, APPLICATION, AND DISPOSAL.

11. THE PERMITTEE MUST PROPERLY MANAGE HAZARDOUS WASTES, USED OILS, CONTAMINATED SOILS, CONCRETE WASTE, SANITARY WASTE, LIQUID WASTE, OR OTHER TOXIC SUBSTANCES DISCOVERED OR GENERATED DURING CONSTRUCTION.

12. THE APPLICATION RATE OF FERTILIZERS USED TO REESTABLISH VEGETATION MUST FOLLOW MANUFACTURER'S RECOMMENDATIONS. NUTRIENT RELEASES FROM FERTILIZERS TO SURFACE WATERS MUST BE MINIMIZED. TIME RELEASE FERTILIZERS SHOULD BE USED AND CARE SHOULD BE MADE IN APPLICATION OF FERTILIZERS WITHIN ANY WATER WAY RIPARIAN ZONE.

13. OWNER OR DESIGNATED PERSON SHALL BE RESPONSIBLE FOR PROPER INSTALLATION AND MAINTENANCE OF ALL EROSION AND SEDIMENT CONTROL MEASURES, IN ACCORDANCE WITH CURRENT CLEAN WATER SERVICES STANDARDS AND STATE, AND FEDERAL REGULATIONS.

14. PRIOR TO ANY LAND DISTURBING ACTIVITIES, THE BOUNDARIES OF THE CLEARING LIMITS, VEGETATED BUFFERS, AND ANY SENSITIVE AREAS SHOWN ON THIS PLAN SHALL BE CLEARLY DELINEATED IN THE FIELD. UNLESS OTHERWISE APPROVED, NO DISTURBANCE IS PERMITTED BEYOND THE CLEARING LIMITS. THE OWNER/PERMITTEE MUST MAINTAIN THE DELINEATION FOR THE DURATION OF THE PROJECT. NOTE: VEGETATED CORRIDORS TO BE DELINEATED WITH ORANGE CONSTRUCTION FENCE OR APPROVED EQUAL.

15. PRIOR TO ANY LAND DISTURBING ACTIVITIES, THE BMPS THAT MUST BE INSTALLED ARE GRAVEL CONSTRUCTION ENTRANCE, PERIMETER SEDIMENT CONTROL, AND INLET PROTECTION. THESE BMPS MUST BE MAINTAINED FOR THE DURATION OF THE PROJECT.

16. IF VEGETATIVE SEED MIXES ARE SPECIFIED, SEEDING MUST TAKE PLACE NO LATER THAN SEPTEMBER 1ST; THE TYPE AND PERCENTAGES OF SEED IN THE MIX ARE AS IDENTIFIED ON THE PLANS OR AS SPECIFIED BY THE DESIGN ENGINEER.

17. WATER-TIGHT TRUCKS MUST BE USED TO TRANSPORT SATURATED SOILS FROM THE CONSTRUCTION SITE. AN APPROVED EQUIVALENT IS TO DRAIN THE SOIL ON SITE AT A DESIGNATED LOCATION USING APPROPRIATE BMPS; SOIL MUST BE DRAINED SUFFICIENTLY FOR MINIMAL SPILLAGE.

18. ALL PUMPING OF SEDIMENT LADEN WATER MUST BE DISCHARGED OVER AN UNDISTURBED, PREFERABLY VEGETATED AREA, AND THROUGH A SEDIMENT CONTROL BMP (I.E. FILTER BAG).

19. THE ESC PLAN MUST BE KEPT ONSITE. ALL MEASURES SHOWN ON THE PLAN MUST BE INSTALLED PROPERLY TO ENSURE THAT SEDIMENT LADEN WATER DOES NOT ENTER A SURFACE WATER SYSTEM, ROADWAY, OR OTHER PROPERTIES.

20. THE ESC MEASURES SHOWN ON THIS PLAN ARE THE MINIMUM REQUIREMENTS FOR ANTICIPATED SITE CONDITIONS. DURING THE CONSTRUCTION PERIOD, THESE MEASURES SHALL BE UPGRADED AS NEEDED TO MAINTAIN COMPLIANCE WITH ALL REGULATIONS.

21. WRITTEN ESC LOGS ARE SUGGESTED TO BE MAINTAINED ONSITE AND AVAILABLE TO DISTRICT INSPECTORS UPON REQUEST.

22. IN AREAS SUBJECT TO WIND EROSION, APPROPRIATE BMPS MUST BE USED WHICH MAY INCLUDE THE APPLICATION OF FINE WATER SPRAYING, PLASTIC SHEETING, MULCHING, OR OTHER APPROVED MEASURES.

23. ALL EXPOSED SOILS MUST BE COVERED DURING WET WEATHER PERIOD.

DESIGN REVIEW AND CONDITIONAL USE PERMIT, PA-15-05

NO.	DATE	REVISION	BY

PRELIMINARY ONLY
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MAY 2015
MURRAY, SMITH & ASSOCIATES, INC. Engineers/Planners

DESIGNED: MLM
DRAWN: DKH
CHECKED: MLM
APPROVED: TPB

SHEET FIG-3 of XXX

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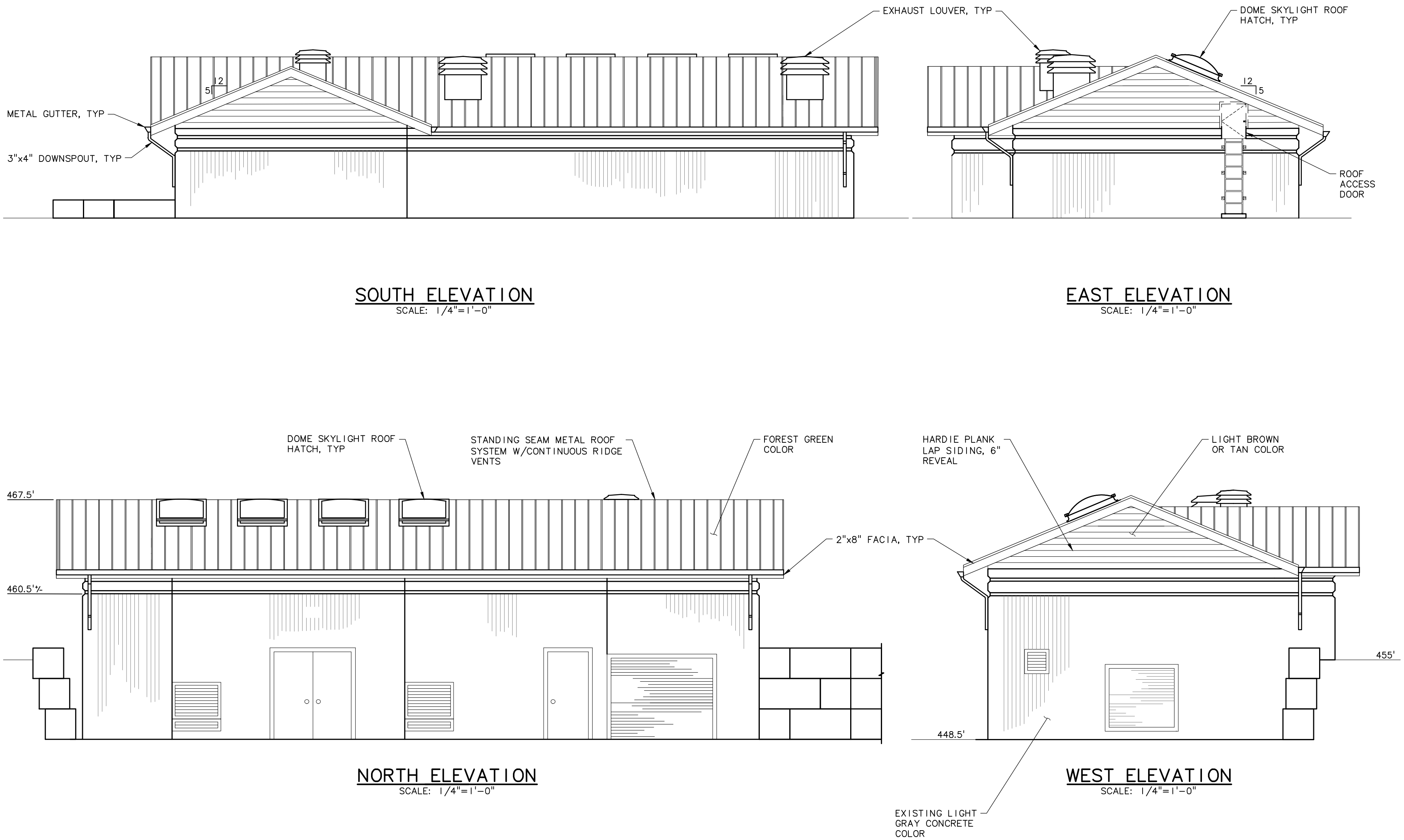
PROJECT NAME: CITY OF WEST LINN, OREGON
BOLTON RESERVOIR REPLACEMENT
PROJECT NO. PW 14-06

SHEET TITLE: EROSION CONTROL MEASURES

MSA Murray, Smith & Associates, Inc. Engineers/Planners
121 S.W. Salmon, Suite 900 Portland, Oregon 97204
PHONE 503-225-9010 FAX 503-225-9022

MSA PROJECT: 14-1586 DATE: MAY 2015

NOTE: PROPOSED NEW ROOF FOR EXISTING PUMP STATION.



DESIGN REVIEW AND CONDITIONAL USE PERMIT, PA-15-05

NO. | DATE | REVISION | BY

DESIGNED: MLM
DRAWN: DKH
CHECKED: MLM
APPROVED: TPB

SHEET FIG-4 of XXX

PRELIMINARY ONLY
DO NOT USE FOR CONSTRUCTION
MAY 2015
MURRAY, SMITH & ASSOCIATES, INC. Engineers/Planners

SCALE VERT: AS SHOWN HORIZ: AS SHOWN
NOTICE
0 1"
IF THIS BAR DOES NOT MEASURE 1" THEN DRAWING IS NOT TO SCALE

PROJECT NAME: CITY OF WEST LINN, OREGON BOLTON RESERVOIR REPLACEMENT PROJECT NO. PW 14-06

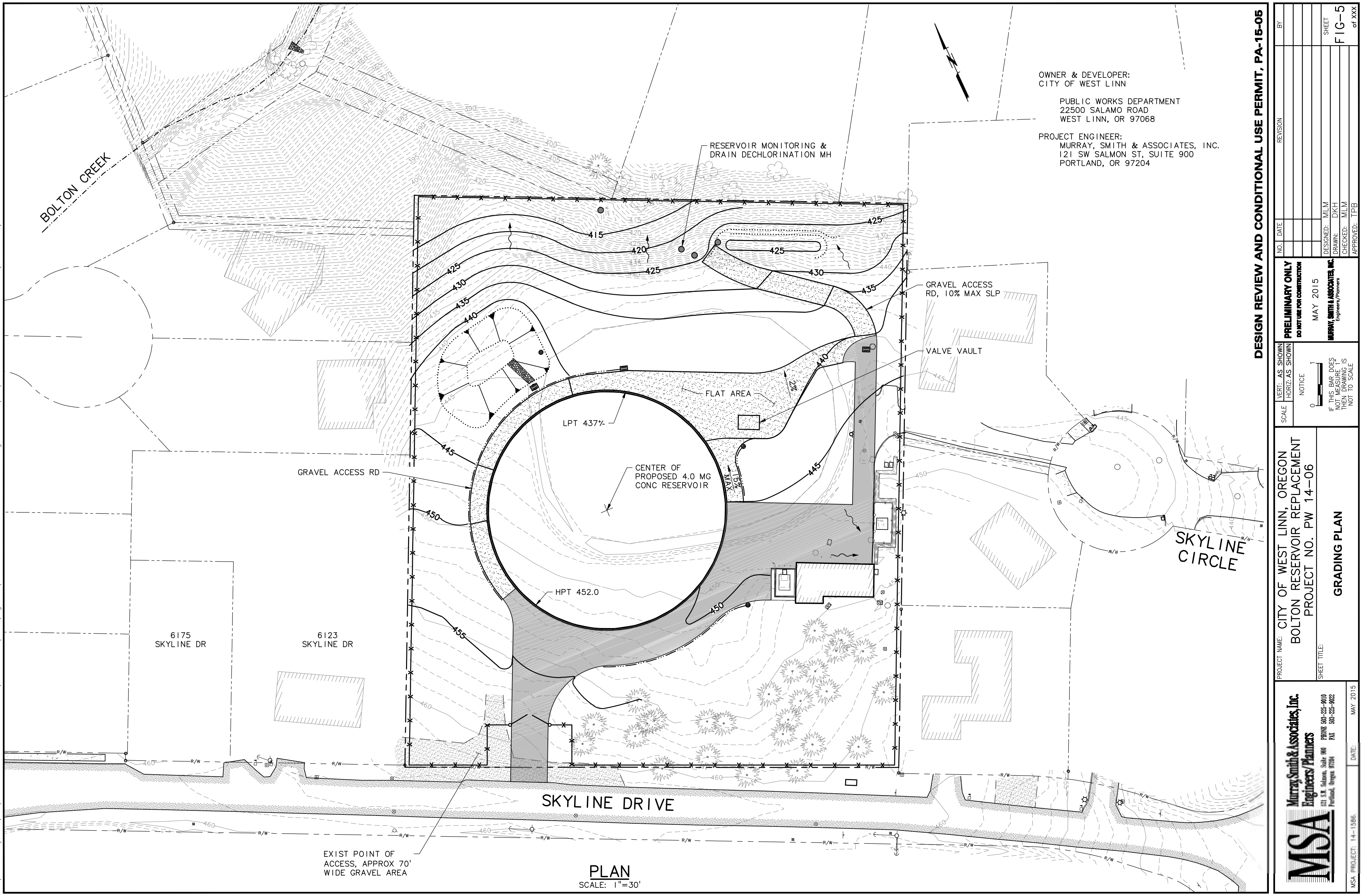
SHEET TITLE: **ARCHITECTURAL ELEVATIONS**

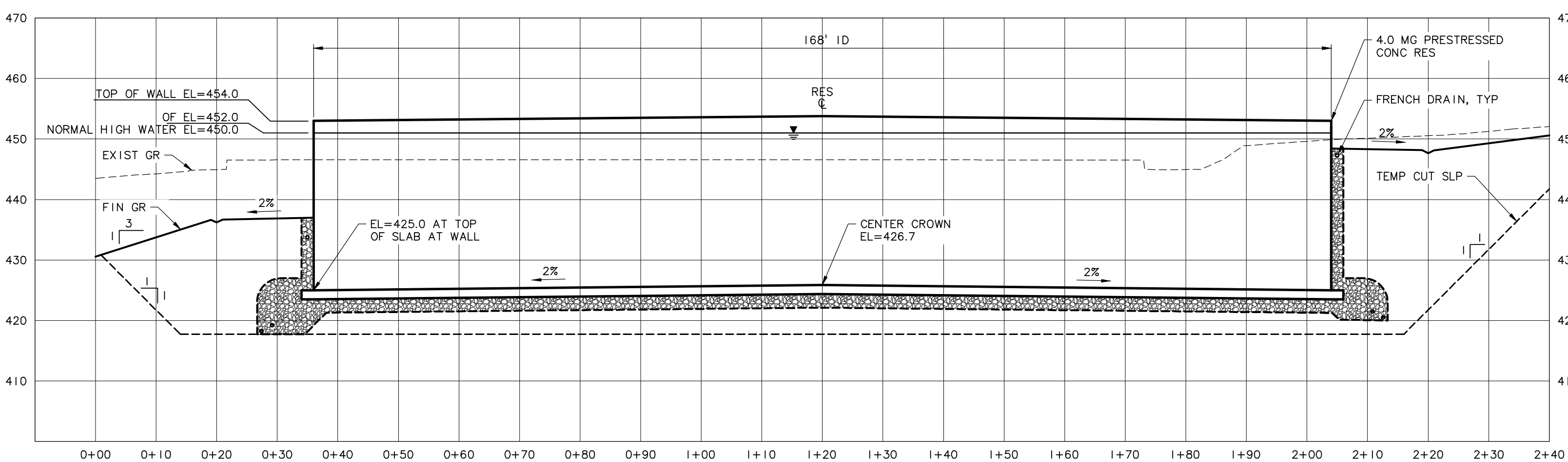
MSA Murray, Smith & Associates, Inc. Engineers/Planners
121 S.W. Salmon, Suite 900 Portland, Oregon 97204 PHONE 503-225-9010 FAX 503-225-9022

MSA PROJECT: 14-1586. DATE: MAY 2015

G:\PDX_Projects\14\1586 - Bolton Reservoir Replacement\CAD\Figures\14-1586-201-OR-CUP FIGS.dwg FIG-4 5/21/2015 1:50 PM DKT 20.0s (LMS Tech)

G:\PDX_Projects\14\1586 - Bolton Reservoir Replacement\CAD\Figures\14-1586-201-OR-CUP FIGS.dwg FIG-5 5/21/2015 1:50 PM DKT 20.0s (LMS Tech)





SECTION

SCALE: 1"=10' HORIZ, 1"=10' VERT

DESIGN REVIEW AND CONDITIONAL USE PERMIT, PA-15-05

G:\PDX_Projects\14\1586 - Bolton Reservoir Replacement\CAD\Figures\14-1586-201-OR-CUP FIGS.dwg FIG-6 5/21/2015 1:50 PM DKT 20.0s (LMS Tech)

MSA Murray, Smith & Associates, Inc.
Engineers/Planners
121 S.W. Salmon, Suite 900 PHONE 503-225-9010
Portland, Oregon 97204 FAX 503-225-9022

MSA PROJECT: 14-1586. DATE: MAY 2015

PROJECT NAME: CITY OF WEST LINN, OREGON
BOLTON RESERVOIR REPLACEMENT
PROJECT NO. PW 14-06

SHEET TITLE: **CONCRETE RESERVOIR SECTION**

SCALE VERT: AS SHOWN HORIZ: AS SHOWN

NOTICE
0 1"
IF THIS BAR DOES NOT MEASURE 1" THEN DRAWING IS NOT TO SCALE

PRELIMINARY ONLY
DO NOT USE FOR CONSTRUCTION
MAY 2015
MURRAY, SMITH & ASSOCIATES, INC.
Engineers/Planners

NO.	DATE	REVISION	BY

DESIGNED: MLM
DRAWN: DKH
CHECKED: MLM
APPROVED: TPB

SHEET FIG-6 of XXX

PLANT SCHEDULE

SYMBOL	KEY	BOTANICAL NAME	COMMON NAME	SIZE	SPACING	QUANTITY	E/D	COMMENTS
		TREES						
	ACCI	ACER CIRCINATUM	VINE MAPLE	2" CAL.	AS SHOWN	12	D	BRANCHED AT 4'-0"; NARROW FORM
	CADE	CALOCEDRUS DECURRENS	INCENSE CEDAR	6–8FT.	AS SHOWN	26	E	
	PSME	PSEUDOTSUGA MENZIESII	DOUGLAS FIR	6–8FT.	AS SHOWN	16	E	
	QUGA	QUERCUS GARRYANA	OREGON WHITE OAK	2" CAL.	AS SHOWN	6	D	
	THPL	THUJA PLICATA	WESTERN RED CEDAR	6–8FT.	AS SHOWN	16	E	
		SHRUBS						
	MAAQ	MAHONIA AQUIFOLIUM	OREGON GRAPE	#1	36" O.C.	118	E	
	POMU	POLYSTICHUM MUNITUM	SWORDFERN	#1	36" O.C.	118	E	
	GASH	GAULTERHIA SHALLON	SALAL	#1	36" O.C.	118	E	

PLANT SCHEDULE: WATER QUALITY SWALE (1,632SF)

SYMBOL	KEY	BOTANICAL NAME	COMMON NAME	SIZE	SPACING	QUANTITY	E/D	COMMENTS
		BOTTOM OF SWALE (459 SF)						
		HERBACEOUS						
	JUPA	JUNCUS PATENS	SPREADING RUSH	4" POT	12" O.C.	264	E	PLANT IN SAME SPECIES GROUPS OF 5-7 PLANTS
	SCMI	SCIRPUS MICROCARPUS	SMALL FRUITED BULRUSH	4" POT	12" O.C.	264	E	
		SIDE SLOPES (1,173SF)						
		TREES						
	ACCI	ACER CIRCINATUM	VINE MAPLE	#2/2' HT.	SINGLE	4	D	MULTI-STEM
		SHRUBS						
	COSE	CORNUS SERICEA 'KELSEY'	KELSEY DOGWOOD	#1	24" O.C.	59	D	PLANT IN SAME SPECIES GROUPS OF 5-7 PLANTS
	RONU	ROSA NUTKANA	NOOTKA ROSE	#1	48" O.C.	58	D	
		GROUNDCOVER						
	MARE	MAHONIA REPENS	CREEPING MAHONIA	4" POT	36" O.C.	821	E	
		SEED						
		HYDROSEED PROTIME 840	NATIVE BIO-FILTER MIX	1 LB/1000 SF				

PLANT SCHEDULE: WATER QUALITY DETENTION POND (4,329SF)

SYMBOL	KEY	BOTANICAL NAME	COMMON NAME	SIZE	SPACING	QUANTITY	E/D	COMMENTS
		TREATMENT AREA (635 SF)						
		HERBACEOUS						
	JUPA	JUNCUS PATENS	SPREADING RUSH	PLUGS	6/SF	1,905	E	PLANT IN SAME SPECIES GROUPS OF 5-7 PLANTS
	SCMI	SCIRPUS MICROCARPUS	SMALL FRUITED BULRUSH	PLUGS	6/SF	1,905	E	
		SIDE SLOPES (3,694SF)						
		TREES						
	ACCI	ACER CIRCINATUM	VINE MAPLE	#2/2' HT.	SINGLE	18	D	MULTI-STEM
	FRLA	FRAXINUS LATIFOLIA	OREGON ASH	#2/3' HT.	SINGLE	19	D	
		SHRUBS						
	COSE	CORNUS SERICEA	RED-TWIG DOGWOOD	#1	36" O.C.	92	D	PLANT IN SAME SPECIES GROUPS OF 5-7 PLANTS
	RONU	ROSA NUTKANA	NOOTKA ROSE	#1	48" O.C.	93	D	
		SEED						
		HYDROSEED PROTIME 840	NATIVE BIO-FILTER MIX	1 LB/1000 SF				

LEGEND

- WATER QUALITY PLANTINGS
- PLANTING AREAS
- ROUGH SEED
- REMOVE EXISTING UNDERGROWTH VEGETATION AND PLACE 2" BARK MULCH BENEATH EXISTING TREES TO REMAIN
- EXISTING TREES TO REMAIN
- ORNAMENTAL FENCE
- CHAINLINK FENCE

WATER QUALITY PLANTING REQUIREMENTS
(Per LIDA Standards)

VEGETATED SWALE:

TOTAL AREA=1,632SF

BOTTOM OF SWALE=459SF
PLANTS REQUIRED: 115 herbaceous/100SF=528

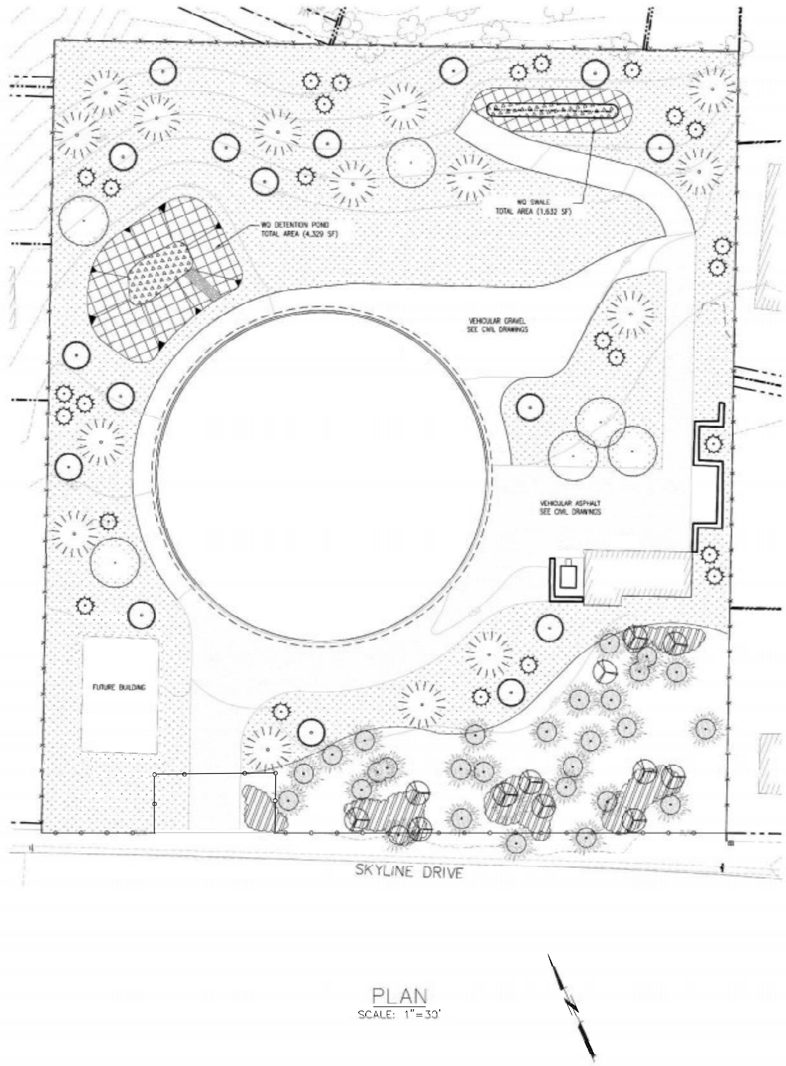
SIDE SLOPES=1,173sf
PLANTS REQUIRED: 1 tree/300SF=4
10 shrubs/100SF=117
70 groundcover/100SF=821

DETENTION POND:

TOTAL AREA=4,329SF

TREATMENT AREA=635SF
PLANTS REQUIRED:6 plugs/SF=3,810

SIDE SLOPES=3,694SF
TREES REQUIRED: area(SF) x 0.01=37
SHRUBS REQUIRED: area(SF) x 0.05=185
GROUNDCOVER REQUIRED: plant and seed to achieve 100% coverage



PLAN
SCALE: 1"=30'

CONDITIONAL USE AND DESIGN REVIEW, PA-15-05

NO.	DATE	REVISION	BY

DESIGNED:
DRAWN:
CHECKED:
APPROVED:

SCALE: VERT: HORIZ:

NOTICE
0 1"
IF THIS BAR DOES NOT MEASURE 1" THEN DRAWING IS NOT TO SCALE

PROJECT NAME: CITY OF WEST LINN, OREGON
BOLTON RESERVOIR REPLACEMENT
PROJECT NO. PW 14-06

SHEET TITLE: LANDSCAPE PLAN

SHEET
FIGURE 7

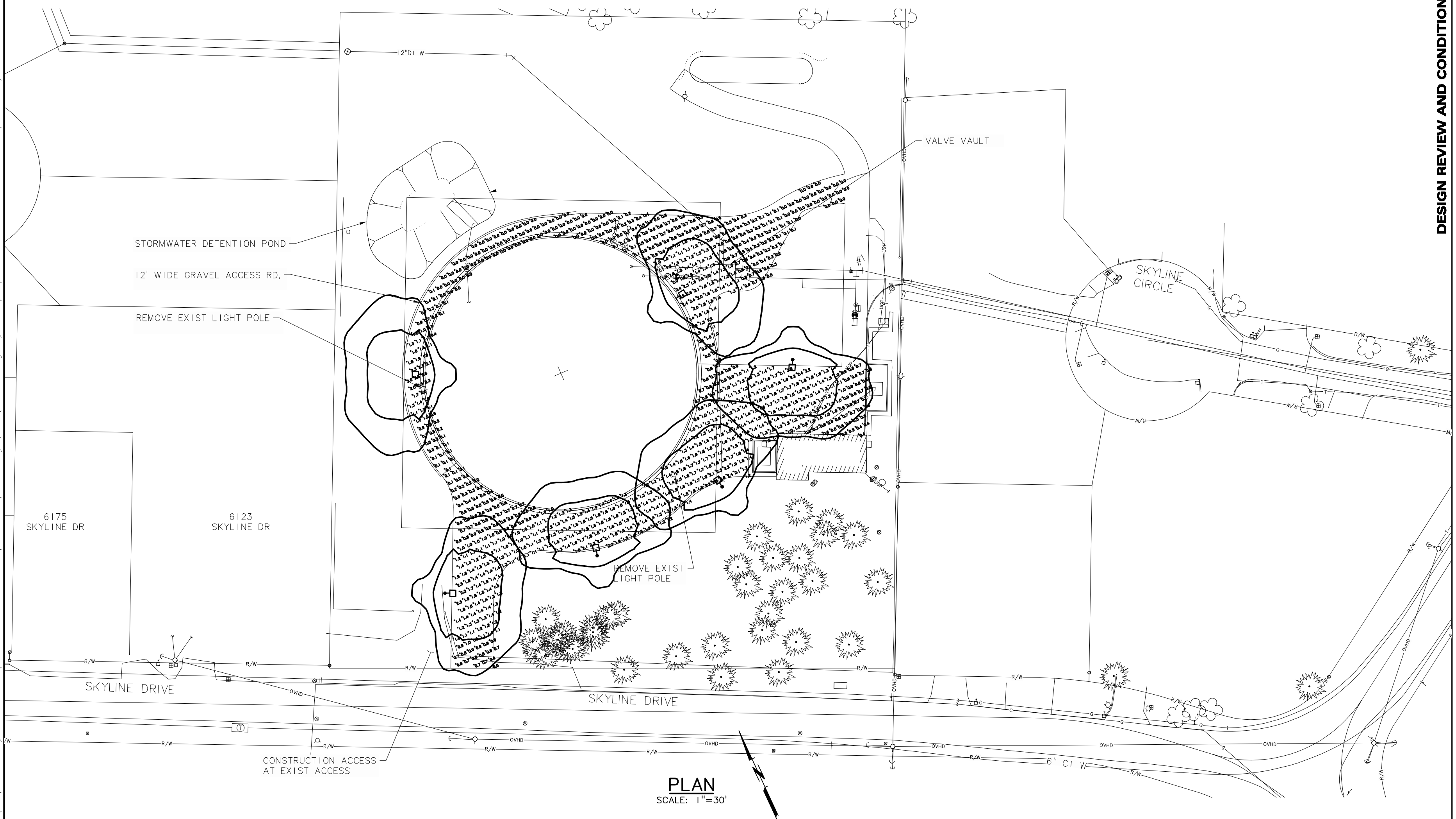
MSA Murray, Smith & Associates, Inc.
Engineers/Planners

MSA PROJECT:
DATE:

STATISTICS							TARGET VALUES	
DESCRIPTION	SYMBOL	AVG	MAX	MIN	MAX/MIN	AVG/MIN	AVG	MAX/MIN
CALC ZONE INTERSECT NORTH	+	1.2 FC	2.8 FC	0.3 FC	9.3:1	4.0:1	≥1.0	≤ 20.0:1

LUMINAIRE SCHEDULE

FIXTURE TYPE	DESCRIPTION	LAMP TYPE	DRIVER	MANUFACTURER AND AND MODEL NUMBER	INPUT WATTS
'SA'	POLE MOUNTED LED LUMINAIRE, 40 LED LIGHT ENGINE, 700ma, 4000K, TYPE 3 DISTRIBUTION, FULL CUT OFF, ACRYLIC LENS, MULTI-VOLT DRIVER, DIE CAST ALUMINUM HOUSING. FINISH DARK BRONZE TO MATCH POLE,	LED 9268 LUMENS	ELECTRONIC DRIVER	LITHONIA LIGHTING: KAD LED SERIES	94W
POLES	20' SQUARE NON TAPERED ALUMINUM POLE. POLE TO WITHSTAND 100 MILE PER HOUR WINDS WITH 1.3 GUST FACTOR. DARK BRONZE FINISH			LITHONIA LIGHTING : SSA	



PLAN
SCALE: 1"=30'

DESIGN REVIEW AND CONDITIONAL USE PERMIT, PA-15-05

W:\WF\483_Murray Smith Assoc\086_Bolton Reservoir Replacement\001_Design\DWG\FIG-07.dwg Layout1 5/13/2015 10:51 AM DAVID 19.1s (LMS Tech)

NO.	DATE	REVISION	BY

DESIGNED: DDH
DRAWN:
CHECKED: GHS
APPROVED:

FIG-8

SCALE VERT: AS SHOWN HORIZ: AS SHOWN

NOTICE
IF THIS BAR DOES NOT MEASURE 1" THEN DRAWING IS NOT TO SCALE

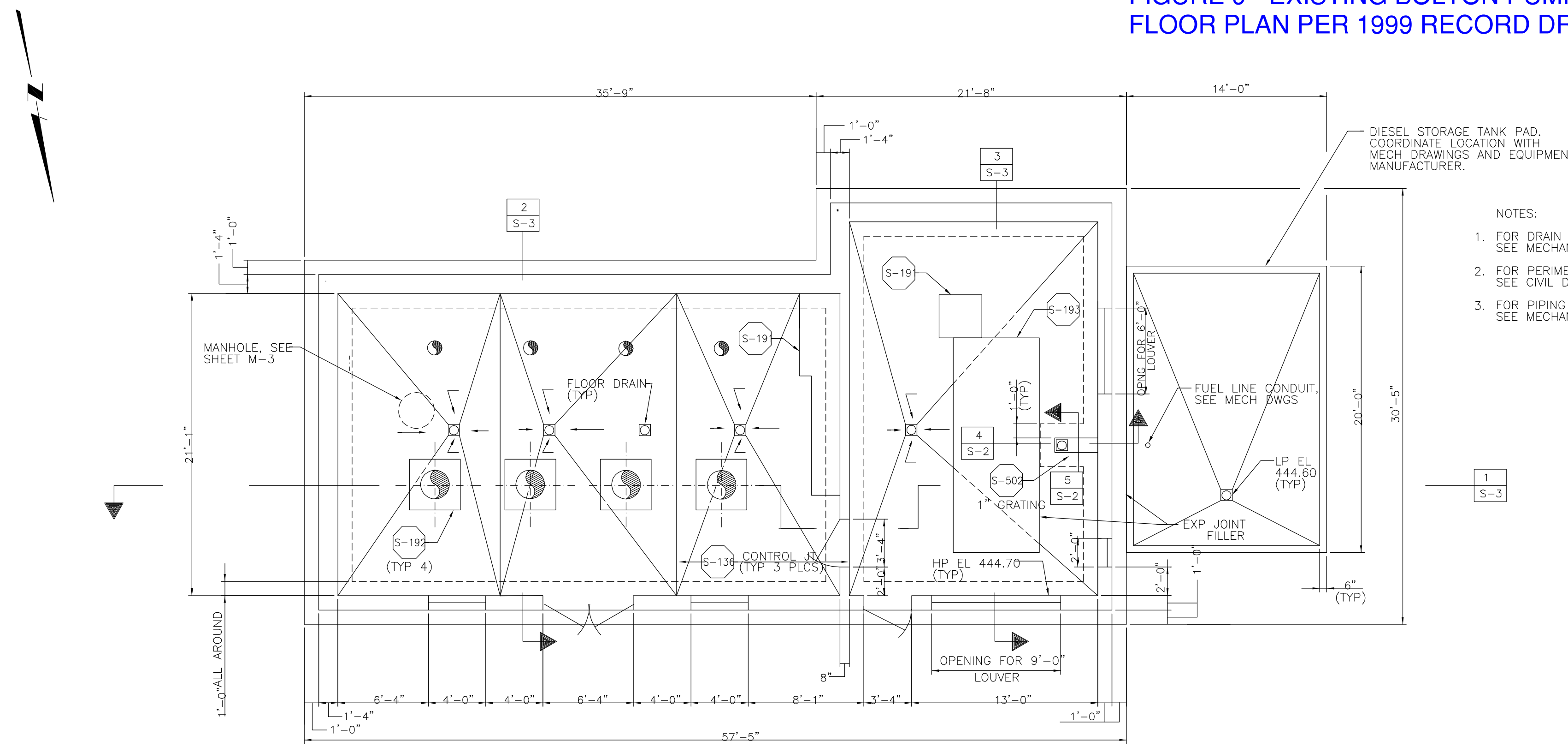
PROJECT NAME: CITY OF WEST LINN, OREGON
BOLTON RESERVOIR REPLACEMENT
PROJECT NO. PW 14-06

SHEET TITLE: LIGHT COVERAGE PLAN

MSA
Murray, Smith & Associates, Inc.
Engineers/Planners
121 S.W. Salmon, Suite 900 PHONE 503-225-9010
Portland, Oregon 97204 FAX 503-225-9022

MSA PROJECT: 14-1586
DATE: MAY 2015

FIGURE 9 - EXISTING BOLTON PUMP STATION FLOOR PLAN PER 1999 RECORD DRAWINGS



FLOOR PLAN

NOTES:

1. FOR DRAIN AND EQUIP PAD LOCATIONS, SEE MECHANICAL DRAWINGS
2. FOR PERIMETER DRAINS, SEE CIVIL DRAWINGS
3. FOR PIPING AND FLOOR PENETRATIONS, SEE MECHANICAL DRAWINGS

This record drawing has been prepared, in part, on the basis of information compiled and furnished by others. The Engineer is not responsible for any errors or omissions which have been incorporated into this document as a result.

RECORD DRAWING
SEE ORIGINAL CONTRACT DRAWINGS FOR SEAL AND SIGNATURES

Plot Date/Time: $$$$date$$$$ $$time$$

FILE: $$$$$$$$$$$$$$$$$$$$directory/filename$$$$$$$$$$$$$$$$$$$

JOB No. 1065073.013584

REV	DATE	BY	DESCRIPTION
	4/99	DPR	RECORD DRAWING

SCALE: 1/4 " = 1'-0"

WARNING: 0 1/2 1 — IF THIS BAR DOES NOT MEASURE 1" THEN DRAWING IS NOT TO SCALE.

DESIGNED J. TEHANEY
DRAWN D. PALMER
CHECKED C. LINDOW

SUBMITTED
PROJECT ENGINEER R. C. E. NO. DATE
MONTGOMERY WATSON R. C. E. NO. DATE

MONTGOMERY WATSON
Portland, Oregon

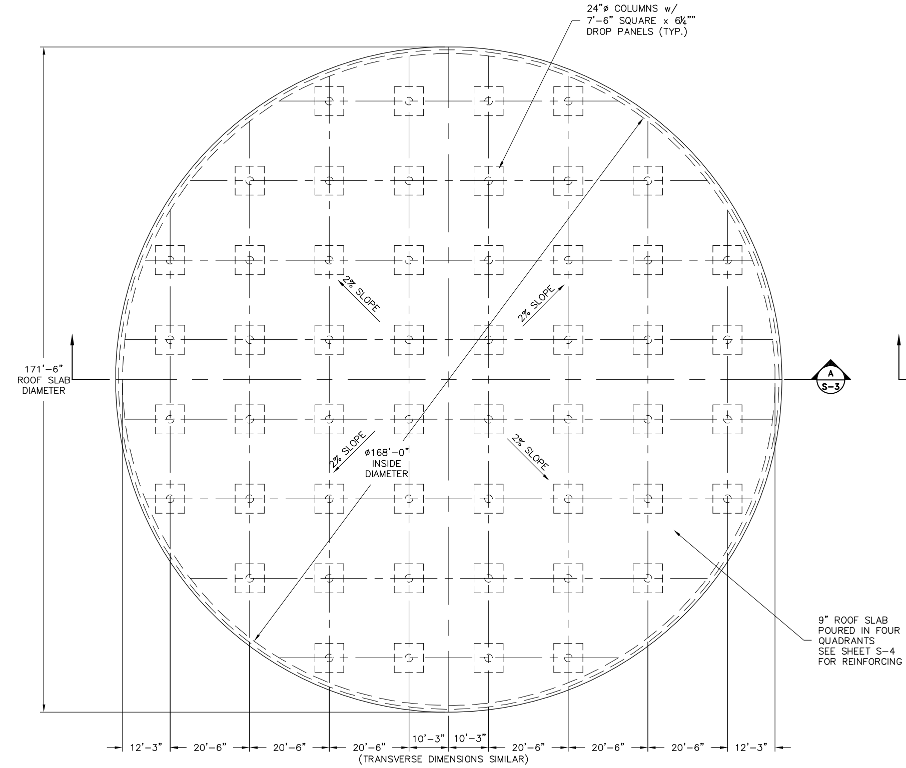
APPROVED DATE
APPROVED DATE

CITY OF WEST LINN
BOLTON PUMP STATION IMPROVEMENTS
PUMP STATION FLOOR PLAN

SHEET S-1 OF 40 SHEETS

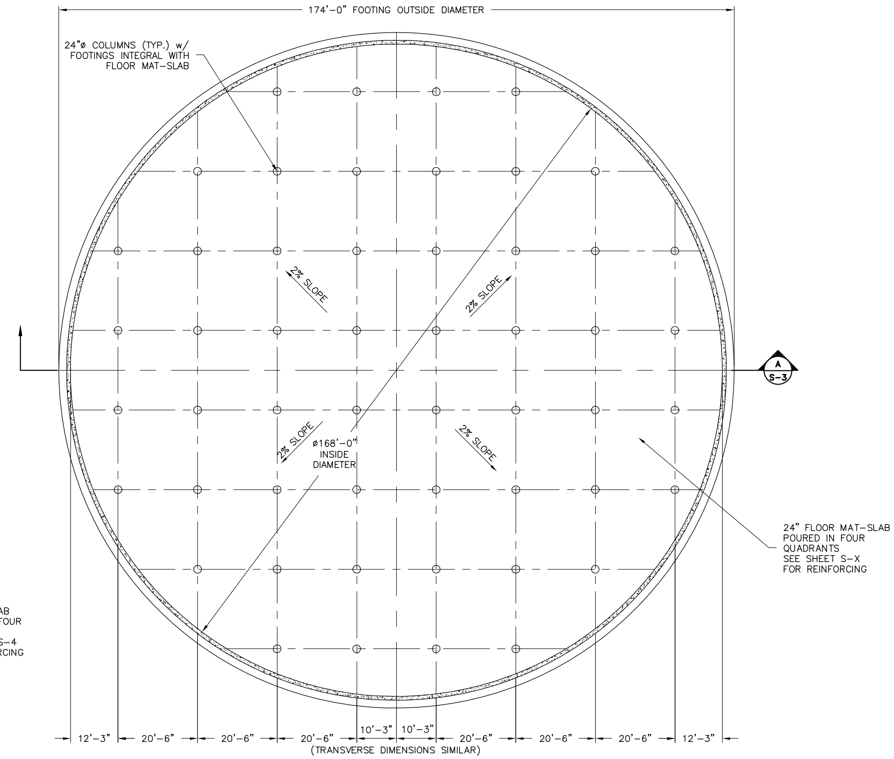
FIGURE 10 - PROPOSED REPLACEMENT RESERVOIR FLOOR AND ROOF PLAN

DESIGN REVIEW AND CONDITIONAL USE PERMIT, PA-15-05



ROOF SLAB PLAN 1/S-2

1/16" = 1'-0"



FLOOR AND FOUNDATION PLAN 2/S-2

1/16" = 1'-0"

NOTE:
SEE CIVIL DRAWINGS FOR PIPING LAYOUT AND DETAILS

PSE
Peterson Structural Engineers, Inc.
5319 S.W. Westgate Dr., Suite 215
Portland, Oregon 97221
(503) 292-1635 1/12/15

NO.	DATE	BY	REVISION

NOTICE

0 ½ 1

IF THIS BAR DOES NOT MEASURE 1" THEN DRAWING IS NOT TO SCALE

EWBP DESIGNED

EWBP DRAWN

TGM CHECKED

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DO NOT USE FOR CONSTRUCTION

JANUARY 2015

MURRAY, SMITH & ASSOCIATES, INC.
Engineers/Planners

MSA Murray, Smith & Associates, Inc.
Engineers/Planners
121 S.W. Salmon, Suite 900
Portland, Oregon 97204
PHONE 503-225-9010
FAX 503-225-9022

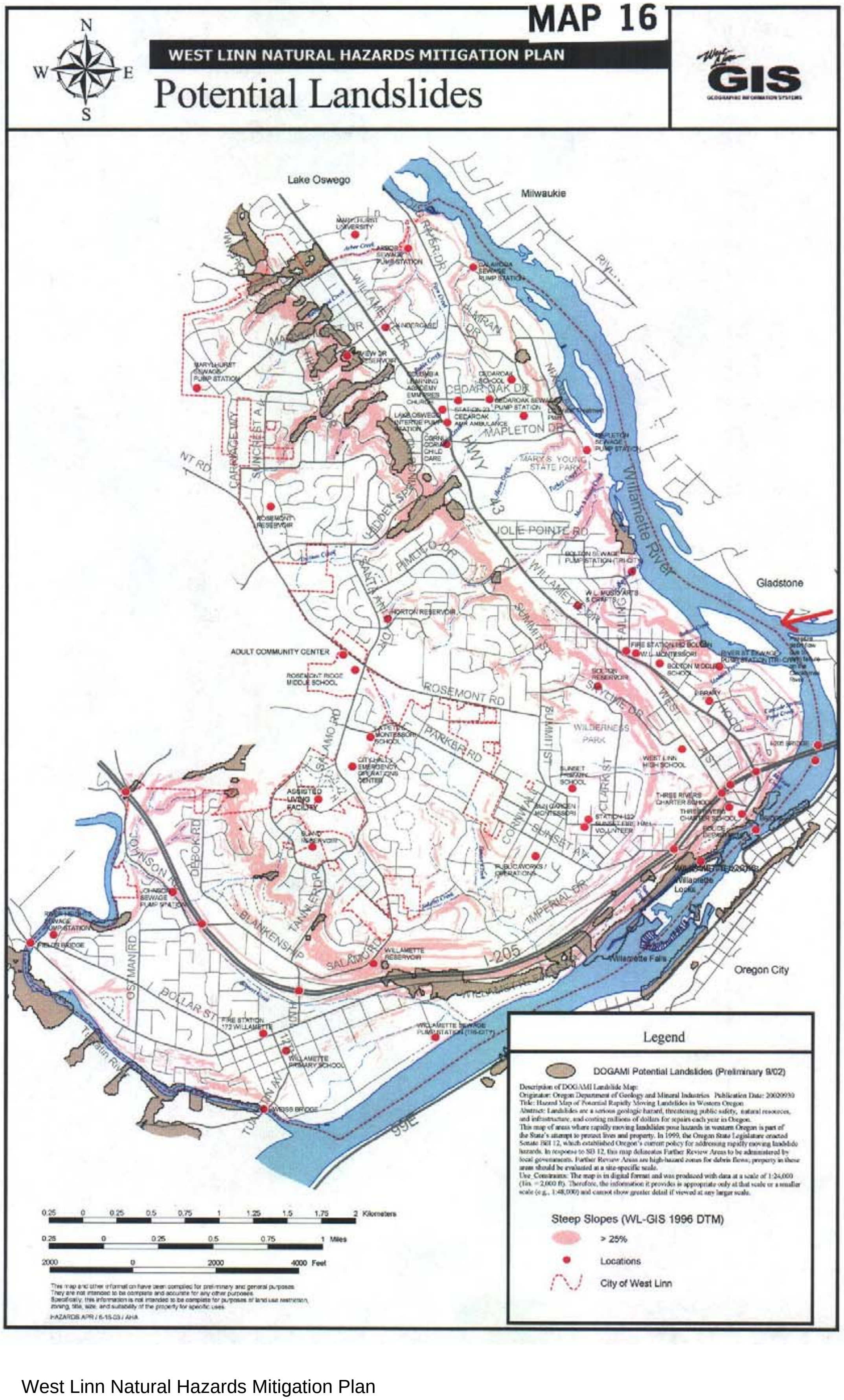
CITY OF WEST LINN
BOLTON RESERVOIR REPLACEMENT

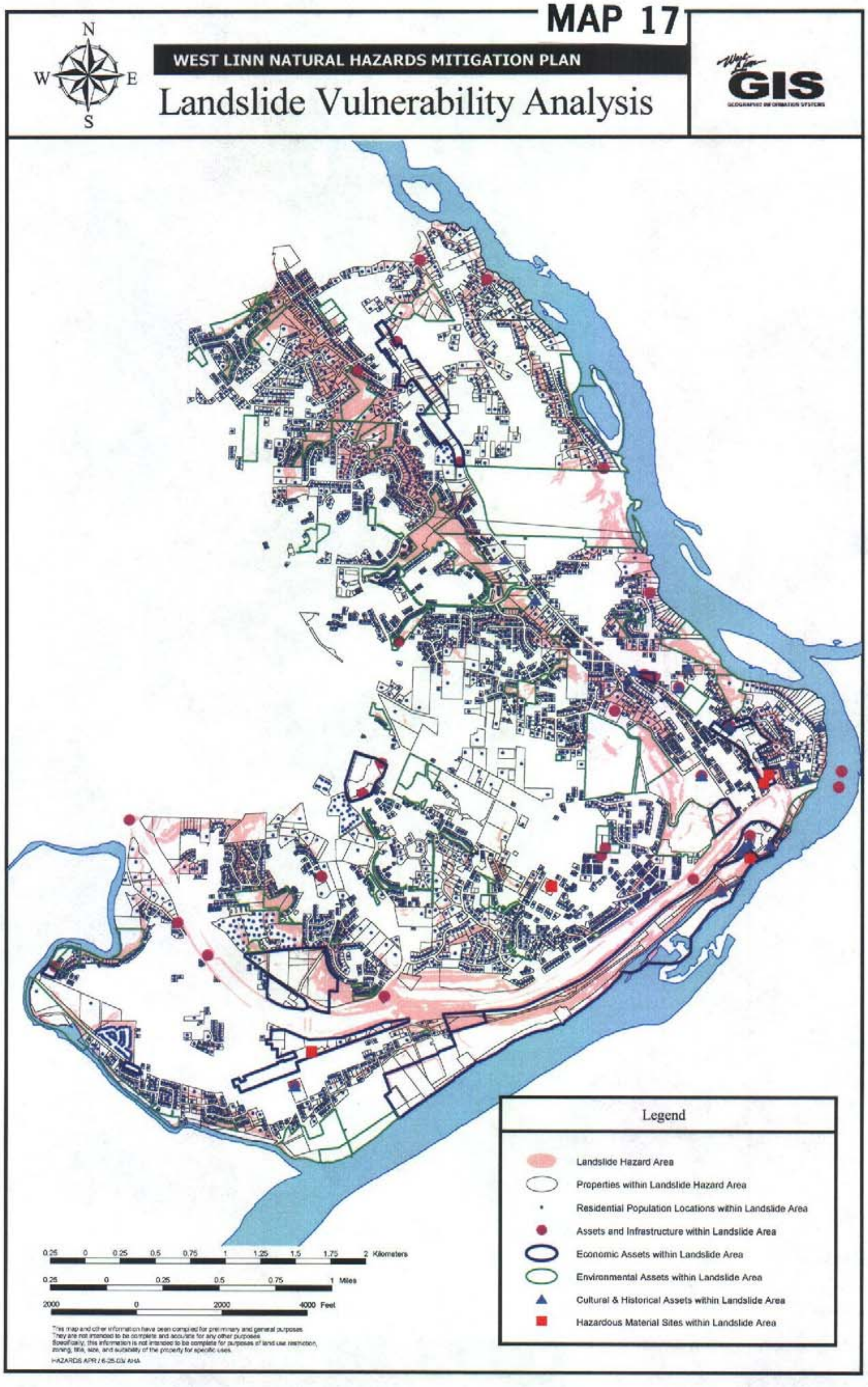
RESERVOIR ROOF AND FLOOR PLANS

SHEET S-2

1 of 2

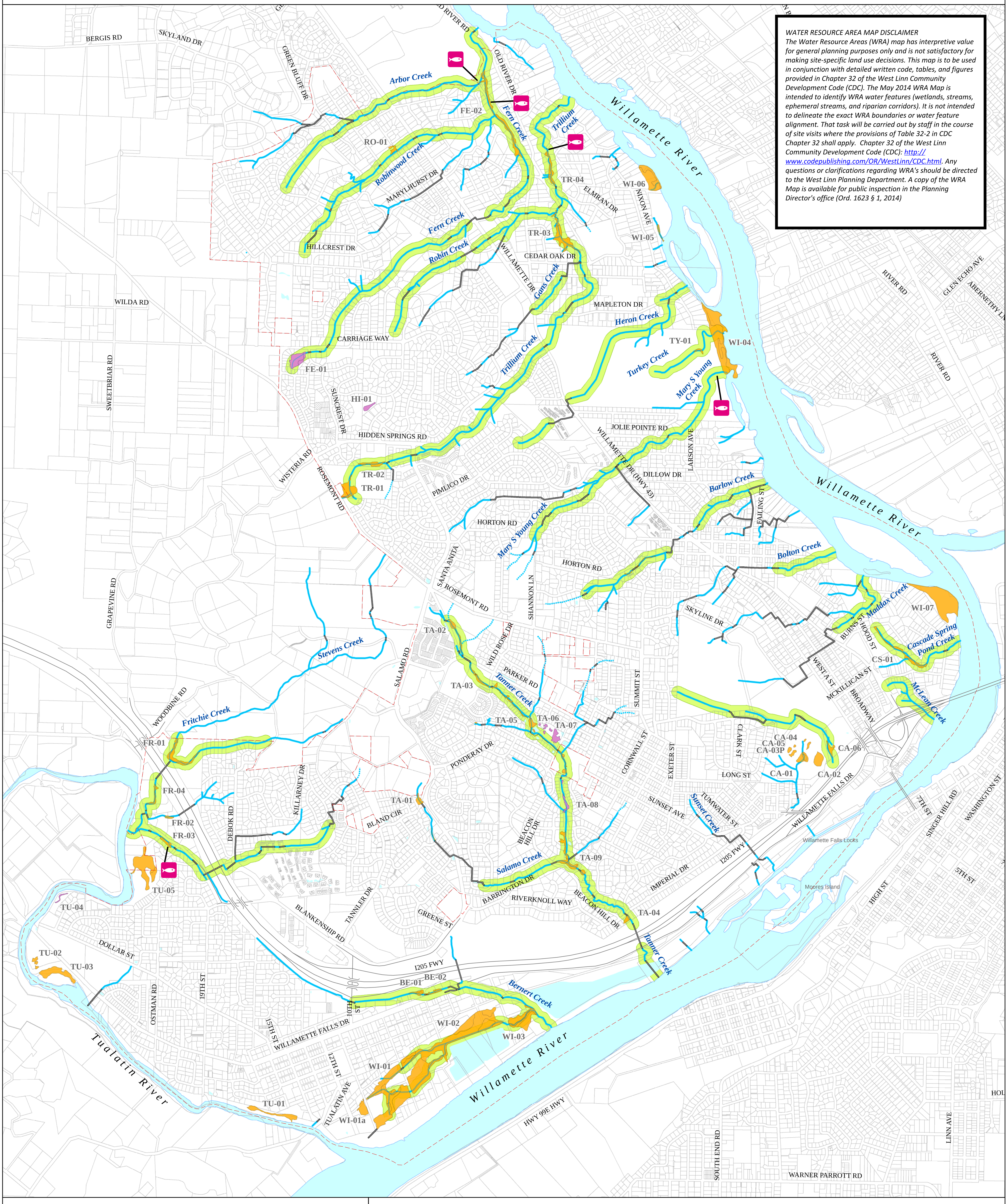
PROJECT NO.: XXXXXXXXXX | SCALE: AS SHOWN | DATE: JANUARY 2015


MAP 16
WEST LINN NATURAL HAZARDS MITIGATION PLAN
Potential Landslides
GIS
Lake Oswego
Milwaukie
Gladstone
Oregon City
Willamette River
ADULT COMMUNITY CENTER
ROSEMONT RD
WILDERNESS PARK
WEST LINN HIGH SCHOOL
I-205
Legend
DOGAMI Potential Landslides (Preliminary 9/02)
Description of DOGAMI Landslide Map:
Originator: Oregon Department of Geology and Mineral Industries Publication Date: 20020930
Title: Hazard Map of Potential Rapidly Moving Landslides in Western Oregon
Abstract: Landslides are a serious geologic hazard, threatening public safety, natural resources, and infrastructure, and costing millions of dollars for repairs each year in Oregon.
This map of areas where rapidly moving landslides pose hazards in western Oregon is part of the State's attempt to protect lives and property. In 1999, the Oregon State Legislature enacted Senate Bill 12, which established Oregon's current policy for addressing rapidly moving landslide hazards. In response to SB 12, this map delineates Further Review Areas to be administered by local governments. Further Review Areas are high-hazard zones for debris flows; property in these areas should be evaluated at a site-specific scale.
Use_Constraints: The map is in digital format and was produced with data at a scale of 1:24,000 (1in. = 2,000 ft). Therefore, the information it provides is appropriate only at that scale or a smaller scale (e.g., 1:48,000) and cannot show greater detail if viewed at any larger scale.
Steep Slopes (WL-GIS 1996 DTM)
> 25%
Locations
City of West Linn
2 Kilometers
1 Miles
4000 Feet
HAZARDS APR / 6-15-03 / AHA


MAP 17
N
W
E
S
WEST LINN NATURAL HAZARDS MITIGATION PLAN
Landslide Vulnerability Analysis
GIS
Legend
Landslide Hazard Area
Properties within Landslide Hazard Area
Residential Population Locations within Landslide Area
Assets and Infrastructure within Landslide Area
Economic Assets within Landslide Area
Environmental Assets within Landslide Area
Cultural & Historical Assets within Landslide Area
Hazardous Material Sites within Landslide Area
0.25 0 0.25 0.5 0.75 1 1.25 1.5 1.75 2 Kilometers
0.25 0 0.25 0.5 0.75 1 Miles
2000 0 2000 4000 Feet
This map and other information have been compiled for preliminary and general purposes.
They are not intended to be complete and accurate for any other purposes.
Specifically, this information is not intended to be complete for purposes of land use restriction, zoning, title, size, and suitability of the property for specific uses.
HAZARDS APR / 6-25-03/ AHA

MAY 2014

Water Resource Area (WRA) Map



WATER RESOURCE AREA MAP DISCLAIMER

The Water Resource Areas (WRA) map has interpretive value for general planning purposes only and is not satisfactory for making site-specific land use decisions. This map is to be used in conjunction with detailed written code, tables, and figures provided in Chapter 32 of the West Linn Community Development Code (CDC). The May 2014 WRA Map is intended to identify WRA water features (wetlands, streams, ephemeral streams, and riparian corridors). It is not intended to delineate the exact WRA boundaries or water feature alignment. That task will be carried out by staff in the course of site visits where the provisions of Table 32-2 in CDC Chapter 32 shall apply. Chapter 32 of the West Linn Community Development Code (CDC): http://www.codepublishing.com/OR/WestLinn/CDC.html. Any questions or clarifications regarding WRA's should be directed to the West Linn Planning Department. A copy of the WRA Map is available for public inspection in the Planning Director's office (Ord. 1623 § 1, 2014)

Map Developed by West Linn Planning Department and GIS

MAP OVERLAYS:
*Streams, Pipe Segments, Other Open Ditches, and Significant Riparian Corridors
Map Source: "Significant Riparian Corridors West Linn Goal 5 Inventory, January 2007"
Map publication date: 1/2/2007.
Modified Streams and added Ephemeral Streams, April 2013, July 2013, September 2013

**Locally Significant Wetlands and Other Wetlands
Map Source: "Local Wetland Inventory, West Linn Goal 5 Inventory, January 2005"
Map publication date: 6/5/2006.

***Taxlot Base Map provided by Clackamas County GIS, 2013

WETLANDS/GOAL 5 DISCLAIMER (DSL STANDARD):
Information shown on this map is for planning purposes only and wetland information is subject to change. There may be unmapped wetlands subject to regulation and all wetland boundary mapping is approximate. In all cases, actual field conditions determine wetland boundaries. You are advised to contact the Oregon Division of State Lands and the U.S. Army Corps of Engineers with any regulatory questions.

This product is for informational purposes and may not have been prepared for, or be suitable for legal, engineering, or surveying purposes. Users of this information should review or consult the primary data and information sources to ascertain the usability of this information.

Goal 5 Significant Riparian Corridors*

Significant Riparian Corridors

Streams

Ephemeral Stream

Piped Segments

Upper Stream Reach of Fish Inventory 2003/2004 Survey

Goal 5 Wetland Inventory**

Locally Significant Wetlands, DSL 2005

Other Wetlands, DSL 2005

TA-05 Specific Wetland Identifier

Rivers & Ponds

West Linn City Limits

Taxlot Base Map***

0 0.25 0.5 1 Miles

Map Created: 6/6/2014

LOC: G:\PROJECTS\GIS\GOAL5_2006\SIGRIPARIAN\SIGRIPARIAN_WETLANDS_201406V6_FINAL.MXD | KAHA
VERSION 5 TO VERSION 6: REMOVED "PROPOSED" FROM MAP TITLE

City of West Linn GIS
GEOGRAPHIC INFORMATION SYSTEMS

City of West Linn

PRE-APPLICATION CONFERENCE MEETING
SUMMARY NOTES

February 05, 2015

SUBJECT: Replace Water Supply reservoir and piping at 6111 Skyline Drive

FILE: PA-15-05

ATTENDEES: Applicants: Lance Calvert, Tom Boland MSA
Staff: John Boyd (Planning), Erich Lais (Engineering)
Other: Alice Richmond, Sally McLarty, Carol Middendorff, Doug Vokes, Alan Lawson, Jennifer Cook-Buman

---

The following is a summary of the meeting discussion provided to you from staff meeting notes. Additional information may be provided to address any "follow-up" items identified during the meeting. These comments are PRELIMINARY in nature. Please contact the Planning Department with any questions regarding approval criteria, submittal requirements, or any other planning-related items. Please note disclaimer statement below.

Site Information

Site Address: 6111 Skyline Blvd
Tax Not No.: Tax Lots 07100 of Assessor's Map 21E25AD
Site Area: 3.23 acres/ 140,700 square feet
Neighborhood: Sunset, Bolton, Hidden Springs/Rosemont Summit
Comp. Plan: Low density residential
Zoning: R-10 (Single family residential detached / 10,000 square foot minimum lot size)
Applicable code: CDC Chapter 60 Conditional Uses
CDC Chapter 55 Design Review
CDC Chapter 11 R-10

Project Details: The applicant proposes replacement of existing covered municipal water supply reservoir with 4.0 million gallon partially buried concrete reservoir. Work will include piping improvements on Skyline Circle and Skyline Drive.

Engineering Division Comments

The applicant should contact Khoi Le of the Engineering Department to determine required improvements at Kle@westlinnoregon.gov. Applicable CDC provisions include Chapter 96.

Process

For the Design Review and Conditional Use Permit, address the submittal requirements and provide responses to the approval criteria of CDC (Chapter 55 & 60.) Municipal projects have no applicable fees.

N/A is not an acceptable response to the approval criteria. The submittal requirements may be waived, but the applicant must first identify the specific submittal requirement and request, in letter form, that it be waived by the Planning Manager and must identify the specific grounds for that waiver.

A neighborhood meeting is required per CDC 99.038. Follow the requirements of that section explicitly. The neighborhood presidents are (Sunset) Tony Breault, available at SunsetNA@westlinnoregon.gov, (Bolton) Sally McLarty, available at BoltonNA@westlinnoregon.gov, and (Hidden Springs/Rosemont Summit) Erik Van de Water, available at HiddenSpringsNA@westlinnoregon.gov

Once the application and deposit/fee are submitted, the City has 30 days to determine if the application is complete or not. If the application is not complete, the applicant has 180 days to make it complete or provide written notice to staff that no other information will be provided. Once the submittal is deemed complete, a hearing with the Planning Commission will be scheduled.

Pre-application notes are void after 18 months. After 18 months with no application approved or in process, a new pre-application conference is required.

Typical land use applications can take 6-10 months from beginning to end.

DISCLAIMER: This summary discussion covers issues identified to date. It does not imply that these are the only issues. The burden of proof is on the applicant to demonstrate that all approval criteria have been met. These notes do not constitute an endorsement of the proposed application ***or provide any assurance of potential outcomes***. Staff responses are based on limited material presented at this pre-application meeting. New issues, requirements, etc. could emerge as the application is developed. ***A new pre-application conference would have to be scheduled one that period lapses and these notes would no longer be valid. Any changes to the CDC standards may require a different design or submittal.***

Oregon
John A. Kitzhaber, MD, Governor

Parks and Recreation Department
State Historic Preservation Office
725 Summer St NE, Ste C
Salem, OR 97301-1266
(503) 986-0690
Fax (503) 986-0793
www.oregonheritage.org

February 13, 2015

RECEIVED
FEB 19 2015
MURRAY, SMITH & ASSOCIATES, INC.

Mr. Michael McKillip
Murray, Smith & Associates, Inc.
121 SW Salmon, Ste 900
Portland, OR 97204-2919

RE: SHPO Case No. 15-0129
City of West Linn, Bolton Resevoir Replacement Project
Remove reservoir & building, install new concrete reservoir
6111 SW Skyline Drive, West Linn, Clackamas County

Dear Mr. McKillip:

We have reviewed the materials submitted on the project referenced above, and we concur with the determination that the property is not eligible for listing in the National Register of Historic Places. We also concur that there will be no historic properties affected for this undertaking.

This letter refers to above-ground historic resources only. Comments pursuant to a review for archaeological resources have been sent separately.

This concludes the requirement for consultation with our office under Section 106 of the National Historic Preservation Act (per 36 CFR Part 800) for above-ground historic properties. Local regulations, if any, still apply and review under local ordinances may be required. Please feel free to contact me if you have any questions, comments or need additional assistance.

Sincerely,

Jason Allen, M.A.
Historic Preservation Specialist
(503) 986-0579
jason.allen@oregon.gov

OREGON SHPO CLEARANCE FORM

Do not use this form for ODOT or Federal Highway projects or to record archaeological sites

*This form is for: **federal** cultural resource reviews (Section 106); **state** cultural resource reviews (ORS 358.653)*

SECTION 1: PROPERTY INFORMATION	**SHPO Case Number:** 15-0129
Resource Name: City of West Linn, Bolton Reservoir	
Street Address: 6111 SW Skyline Drive	
City: West Linn	County: Clackamas
Agency Project # PW-14-06	Project Name: Bolton Reservoir Replacement

If there is not a street address, include the Township, Range, and Section, cross streets, or other address description

Owner: ☐ Private ☒ Local Gov ☐ State Gov ☐ Federal Gov ☐ Other:_________

Are there one or more buildings or structures? ☒ YES ☐ NO – If no, skip to Section 2 and append photo(s)

Is the property listed in the National Register of Historic Places? ☐ YES – Individually ☐ YES – In a district ☒ NO

Original Construction date: 1915 ☐ Check box if date is estimated

Siding Type(s) and Material(s): open-air reservoir Window Type(s) and Material(s):

Has the property been physically altered? ☐ No Alterations ☐ Few Alterations ☒ Major / Many Alterations

SECTION 2: APPLICANT DETERMINATION OF ELIGIBILITY *- Check the appropriate box*

The purpose of this review is to avoid impacts to properties that are "eligible" (historic) or already listed in the National Register of Historic Places. Fully establishing historic significance can be very costly and time consuming. Therefore initial evaluations are based on age (50 years or greater) and integrity (historic appearance), which are the minimum qualifications for listing in the National Register. Additional documentation may be needed further in the process, but typically initial evaluations allow the review process to proceed expeditiously.

☐ The property is considered **Eligible** at this time because it is already listed in the National Register ***or***

- is at least 50 years old ***and*** retains its historic integrity (minimal alterations to key features)
- has potential significance (architectural or historical)

☒ The property is considered **Not Eligible** at this time because it:

- is less than 50 years old ***or*** is 50 years or older but there have been major alterations to key features
- is known to have no significance, based on National Register-level documentation and evaluation

SECTION 3: APPLICANT DETERMINATION OF EFFECT *- Check the appropriate box*

☒ The project has **NO EFFECT** on a property that is eligible or already listed in the National Register, either because there is no eligible property involved or the eligible property will not be impacted physically or visually.

☐ The project will have a minor impact on a property that is eligible or already listed in the National Register, and therefore there is **NO ADVERSE EFFECT**. Minor impacts include replacement of some, but not all, siding, doors, or windows, etc.

☐ The project will have a major impact on a property that is eligible or already listed in the National Register, therefore there is an **ADVERSE EFFECT**. Major impacts include full or partial demolition, complete residing, full window replacement, etc.

STATE HISTORIC PRESERVATION OFFICE COMMENTS – *Official use only*

Eligibility: ☒ Concur with the eligibility determination above.
☐ Do not concur with the eligibility determination above.

Effect: ☒ Concur with the effect determination above.
☐ Do not concur with the effect determination above.

RECEIVED STAMP

RLS	
ILS	

Signed: ______________ Date: 2/13/2015

CONTACT INFORMATION STAMP

JASON ALLEN
503-986-0579
Jason.Allen@oregon.gov

Comments:

OREGON SHPO CLEARANCE FORM

Do not use this form for ODOT or Federal Highway projects or to record archaeological sites

SECTION 4: PREVIOUS ALTERATIONS TO THE BUILDING OR STRUCTURE

Only complete this section for buildings that are 50 years old or older. Describe any alterations that have already occurred to the building, such as material replacement, including siding, windows, and doors; any additions, including garages; and any removal or addition of architectural details, such as brackets, columns, and trim. Provide estimated dates for the work. Attach additional pages as necessary.

Constructed as an open-air water supply reservoir circa 1915. A pump station building was added (date unknown) and subsequently taken out of service and a new pump station was added in the 1980s. Chain link fencing has been added around the reservoir. An interior liner was installed in 1992. A synthetic cover was added in 1996.

SECTION 5: PROJECT DESCRIPTION

Describe what work is proposed, including what materials will be used and how they will be installed. Specifically identify what historic materials will be retained, restored, replaced, or covered. Include drawings, photos, cut sheets (product descriptions), additional sheets, and other materials as necessary. For vacant lots, please describe the intended use.

The original 1915 reservoir is past its useful life and exhibits structural problems. The reservoir is also undersized and does not meet current seismic design standards. The project will remove the existing reservoir and the abandoned pump station building and install a new partially buried pre-stressed concrete reservoir at approximately the same location.

SECTION 6: FUNDING SOURCE

☐ ARRA ☐ FCC ☐ FERC ☐ HUD ☐ ODOE ☐ USDARD ☐ USFS
☒ Other: City of West Linn

SECTION 7: AGENCY CONTACT INFORMATION

Name of Organization Submitting the Project: City of West Linn, Public Works Department

Project Contact Name and Title: Mr. Lance Calvert, Public Works Director

Street Address, City, Zip: 22500 Salamo Drive, West Linn, OR 97068

Phone: 503-722-5516 | Email: LCALVERT@westlinnoregon.gov

Date of Submission: January 26, 2015

SECTION 8: ATTACHMENTS

REQUIRED	☒ 3 – 4, color, 4 x 5 photographs of the subject property, digital or print. One photo is sufficient for vacant property
AS NEEDED *Contact SHPO staff with questions*	☒ Project area map, for projects including more than one tax lot
	☐ Additional drawings, reports, or other relevant materials
	☐ Continuation sheet for sections 4 or 5, or additional context to determine National Register Eligibility.

SHPO Mailing Address: Review and Compliance, Oregon SHPO, 725 Summer St. NE, Suite C, Salem, OR 97301
Documents meeting all aspects of the digital submission policy may be submitted by email to ORSHPO.Clearance@state.or.us

CITY HALL 22500 Salamo Rd, West Linn, OR 97068 Telephone: (503) 657-0331 Fax: (503) 650-9041

City of West Linn

Certified Mail
Return Receipt Requested

March 31, 2015

Anthony Breault, President
1890 Sunset Court
West Linn OR 97068

Doreen Vokes, Secretary/Treasurer
4972 Prospect St.
West Linn OR 97068

Ref: 6111 Skyline Drive
Tax Lot: 21E25AD07100
West Linn, Oregon 97068

Dear Mr. Breault and Ms. Vokes:

The City of West Linn is contacting you regarding the property located at 6111 Skyline Drive in the Sunset Neighborhood Association. A land use permit application for replacement of the drinking water reservoir is planned to be submitted to the City of West Linn. As part of applying to the City of West Linn for the necessary land use approvals, we would like to discuss the project in more detail with the Neighborhood Association, surrounding property owners and residents and to review your concerns. We will make a short presentation and allow time for discussion from interested parties.

We are scheduling a separate public meeting from your regular meeting and will invite the Neighborhood Association and neighbors in the surrounding area to attend, per the City's requirements. The meeting is set for April 23 at 7 p.m. at the West Linn Public Library, 1595 Burns Street, West Linn.

If you have questions, please feel free to call 503-722-5500 or contact me by email at lcalvert@westlinnoregon.gov.

Sincerely,

Lance Calvert
City of West Linn Public Works Director

CITY HALL 22500 Salamo Rd, West Linn, OR 97068 Telephone: (503) 657-0331 Fax: (503) 650-9041

City of West Linn

March 31, 2015

NEIGHBORHOOD MEETING NOTICE

Ref: 6111 Skyline Drive
Tax Lot: 21E25AD07100
West Linn, Oregon 97068

Dear Interested Party:

The City of West Linn is contacting you regarding the property located at 6111 Skyline Drive in the Sunset Neighborhood Association. A land use permit application for a water reservoir is planned to be submitted to the City of West Linn. As part of applying to the City of West Linn for the necessary land use approvals, we would like to discuss the project in more detail with the Neighborhood Association, surrounding property owners and residents. You are invited to attend a scheduled meeting on:

Thursday, April 23, 2015 at 7 p.m.
West Linn Public Library Community Room
1595 Burns Street
West Linn, OR 97068

Please note that this will be an informational meeting on preliminary plans. These plans may be modified before the application is submitted to the City. You may also receive an official notice from the City of West Linn after the application is accepted, advising you of your opportunity to participate in the City process.

I look forward to discussing this project with you. If you have questions, but will be unable to attend, please feel free to call 503-722-5500.

Sincerely,

Lance Calvert
City of West Linn Public Works Director

ANNERY THOMAS MARK & VELMA KATHERINE
6081 CAUFIELD ST
WEST LINN, OR 97068

AYALA MARY E
6025 SKYLINE DR
WEST LINN, OR 97068

BARKER JOEL
6040 CAUFIELD ST
WEST LINN, OR 97068

BEEHLER JOSEPH P & MARIANNE P
6041 CAUFIELD ST
WEST LINN, OR 97068

BELL CRAIG S & JULIANNE
6035 SKYLINE DR
WEST LINN, OR 97068

BORGMEIER TODD S
745 E FAIRFIELD ST
GLADSTONE, OR 97027

BROWN D BRUCE
6005 SKYLINE CIR
WEST LINN, OR 97068

BROWN JOHN MICHAEL & J LAROCHELLE
6292 BRIDGEVIEW DR
WEST LINN, OR 97068

CALVIN JON
2505 SE STUBB ST
MILWAUKIE, OR 97222

CASSAR MARCUS J & KARA A
5053 WOODWINDS CT
WEST LINN, OR 97068

CAUFIELD PARK APARTMENTS LLC
PO BOX 859
MOLALLA, OR 97038

CHANEY PHILIP ARTHUR & HEATHER L
5176 FIRWOOD PL
WEST LINN, OR 97068

CITY OF WEST LINN
22500 SALAMO RD #600
WEST LINN, OR 97068

COOK JENNIFER M
6010 SKYLINE CIR
WEST LINN, OR 97068

CULL JOHN
6042 WEST A ST
WEST LINN, OR 97068

CULSHAW MARY L & MICHAEL S
24390 SW BAKER RD
SHERWOOD, OR 97140

DECLARK JAMES A CO-TRUSTEE
6140 CAUFIELD ST
WEST LINN, OR 97068

DRAYTON TED HENRY
PO BOX 566
WEST LINN, OR 97068

FARLEIGH ERIK N & ERICA A
5127 FIRWOOD DR
WEST LINN, OR 97068

FARRAR STEVEN D
6080 CAUFIELD ST
WEST LINN, OR 97068

FOSTER JAMES CONRAD TRUSTEE
19363 WILLAMETTE DR #135
WEST LINN, OR 97068

FUNG GLENN W & PHAN CAM DANG
4027 IMPERIAL DR
WEST LINN, OR 97068

GIARDINA ANTHONY J & KAREN D
5191 FIRWOOD PL
WEST LINN, OR 97068

GOODMAN LINDA & MARK ENGDALL
6001 CAUFIELD ST
WEST LINN, OR 97068

GORDON LILLIAN M
5163 FIRWOOD DR
WEST LINN, OR 97068

GOWDY ROBERT S
6292 EVERGREEN DR
WEST LINN, OR 97068

GRAGG JOHN DANIEL & JILL B
5110 FIRWOOD DR
WEST LINN, OR 97068

HAYASHI HOWARD H TRUSTEE
6120 CAUFIELD ST
WEST LINN, OR 97068

HOLMAN BRIAN D & CHERI L
5134 FIRWOOD DR
WEST LINN, OR 97068

KERN ANDREW S
5111 FIRWOOD DR
WEST LINN, OR 97068

KILDAHL KRISTEN
6020 CAUFIELD ST
WEST LINN, OR 97068

KNIGHT ALICE O & NIXON A
16562 S CARUS RD
BEAVERCREEK, OR 97004

KOCER DAVID & ANN
2425 WOODHAVEN CT
WEST LINN, OR 97068

KOZIOL JOSEPH W
5990 WEST A ST
WEST LINN, OR 97068

KRAMER WILLIAM B TRUSTEE
6175 SKYLINE DR
WEST LINN, OR 97068

LAM BRENT & ZORAYDA
28300 NE BELL RD
NEWBERG, OR 97132

LARSON RICHARD G & MARY L
5955 SKYLINE DR
WEST LINN, OR 97068

LAWSON ALAN S
6008 SKYLINE CIR
WEST LINN, OR 97068

LESTER DAN & PATRICE
2200 HAMMERLE ST
WEST LINN, OR 97068

MAGNUSON ROBERT & AUDREY
2150 HAMMERLE ST
WEST LINN, OR 97068

MIDDENDORFF STEPHEN CARL
6015 SKYLINE CIR
WEST LINN, OR 97068

MOLINA SERGIO
PO BOX 859
MOLALLA, OR 97038

MORIARTY ERIKO & ROBERT E
5147 FIRWOOD CT
WEST LINN, OR 97068

MULLINS WILLIAM H TRUSTEE
8696 SE 141ST CT
HAPPY VALLEY, OR 97086

MUNIZ SANTIAGO & NANCY E
5183 FIRWOOD PL
WEST LINN, OR 97068

OLSON KENNETH V & JOELLA K N
6021 CAUFIELD ST
WEST LINN, OR 97068

OLUND LARRY S & MARY H
6000 CAUFIELD ST
WEST LINN, OR 97068

ORME MAYNARD E
5190 FIRWOOD PL
WEST LINN, OR 97068

PEARCE DALE R
6061 CAUFIELD ST
WEST LINN, OR 97068

PFAHL BRENDA L TRUSTEE
c/o ALVIN PFAHL
6003 SKYLINE CIR
WEST LINN, OR 97068

PORCHE ANTONIO & BARBARA
2140 HAMMERLE ST
WEST LINN, OR 97068

ROTHENHOEFER JANICE L
6101 CAUFIELD ST
WEST LINN, OR 97068

RUNKEL SCOTT A & PTOLEMY
5151 FIRWOOD CT
WEST LINN, OR 97068

SHUKUR HAMID
6288 BRIDGEVIEW DR
WEST LINN, OR 97068

SILVA SYLVIA
6110 CAUFIELD ST
WEST LINN, OR 97068

SMITH JUSTIN F
25925 S ELDORADO RD
MULINO, OR 97042

SNAPP TERRY G
5128 FIRWOOD DR
WEST LINN, OR 97068

SONTAG ROBERT E
5077 WOODWINDS CT
WEST LINN, OR 97068

TUNSTALL LETHA
5041 WOODWINDS CT
WEST LINN, OR 97068

WITTE JON P & SUSAN L
6100 CAUFIELD ST
WEST LINN, OR 97068

ALAN SMITH
BOLTON NA PRESIDENT
1941 BUCK ST
WEST LINN OR 97068

STEVE MIESEN
BOLTON NA VICE PRESIDENT
6275 HOLMES ST
WEST LINN OR 97068

ERIK VAN DE WATER
HSRS NA PRESIDENT
6433 PALOMINO WAY
WEST LINN OR 97068

DOREEN VOKES
SUNSET NA SEC/TREAS
4972 PROSPECT ST
WEST LINN OR 97068

TONY BREAULT
SUNSET NA PRESIDENT
1890 SUNSET CT
WEST LINN OR 97068

NEIGHBORHOOD MEETING

AFFIDAVIT OF MAILING

CITY OF WEST LINN)

SS

CITY OF WEST LINN)

I, Lori Hall, being duly sworn, state that I represent the party initiating interest in a proposed water reservoir affecting the land located at 6111 Skyline Drive in West Linn, Oregon, and that pursuant to Community Development Code Section 99, did on March 31, 2015, cause to have mailed, to each of the persons on the attached list, a notice of a meeting to discuss the proposed development of the aforementioned property.

I further state that said notices were enclosed in plainly addressed envelopes to said persons and were deposited on the date indicated above in the United States Post Office with postage prepaid thereon.

The 18 day of May, 2015.

Hall

Signature

Subscribed and sworn to, or affirmed, before me this 18TH day of MAY, 2015.

OFFICIAL SEAL
NANCY ANN EVETT
NOTARY PUBLIC - OREGON
COMMISSION NO. 465429
MY COMMISSION EXPIRES JANUARY 31, 2016

Notary Public for the State of Oregon

County of Clackamas

My Commission Expires: January 31, 2015

7012 0470 0000 0124 9339

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3-31-15

Sent To Erik Van der Water
Street, Apt. No.; or PO Box No. 6433 Palomino Way
City, State, ZIP+4 West Linn OR 97068

PS Form 3800, August 2006 See Reverse for Instructions

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Restricted Delivery Fee (Endorsement Required)	
Total Postage & Fees	$ 6.48

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3-31-15

Sent To Doreen Vokes Sunset NA
Street, Apt. No.; or PO Box No. 4972 Prospect St
City, State, ZIP+4 West Linn, OR 97068

PS Form 3800, August 2006 See Reverse for Instructions

7012 0470 0000 0124 9353

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Restricted Delivery Fee (Endorsement Required)	
Total Postage & Fees	$ 6.48

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3-31-15

Sent To Alan Smith BNA
Street, Apt. No.; or PO Box No. 1941 Buck St
City, State, ZIP+4 West Linn OR 97068

PS Form 3800, August 2006 See Reverse for Instructions

7012 0470 0000 0124 9346

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Restricted Delivery Fee (Endorsement Required)	
Total Postage & Fees	$ 6.48

Postmark Here

3-31-15

Sent To Tony Breault SNA
Street, Apt. No.; or PO Box No. 1890 Sunset Ct
City, State, ZIP+4 West Linn OR 97068

PS Form 3800, August 2006 See Reverse for Instructions

000 0124 9377

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Restricted Delivery Fee	

Postmark Here

Notice of Neighborhood Meeting regarding a proposed water reservoir for property located at 6111 Skyline Drive

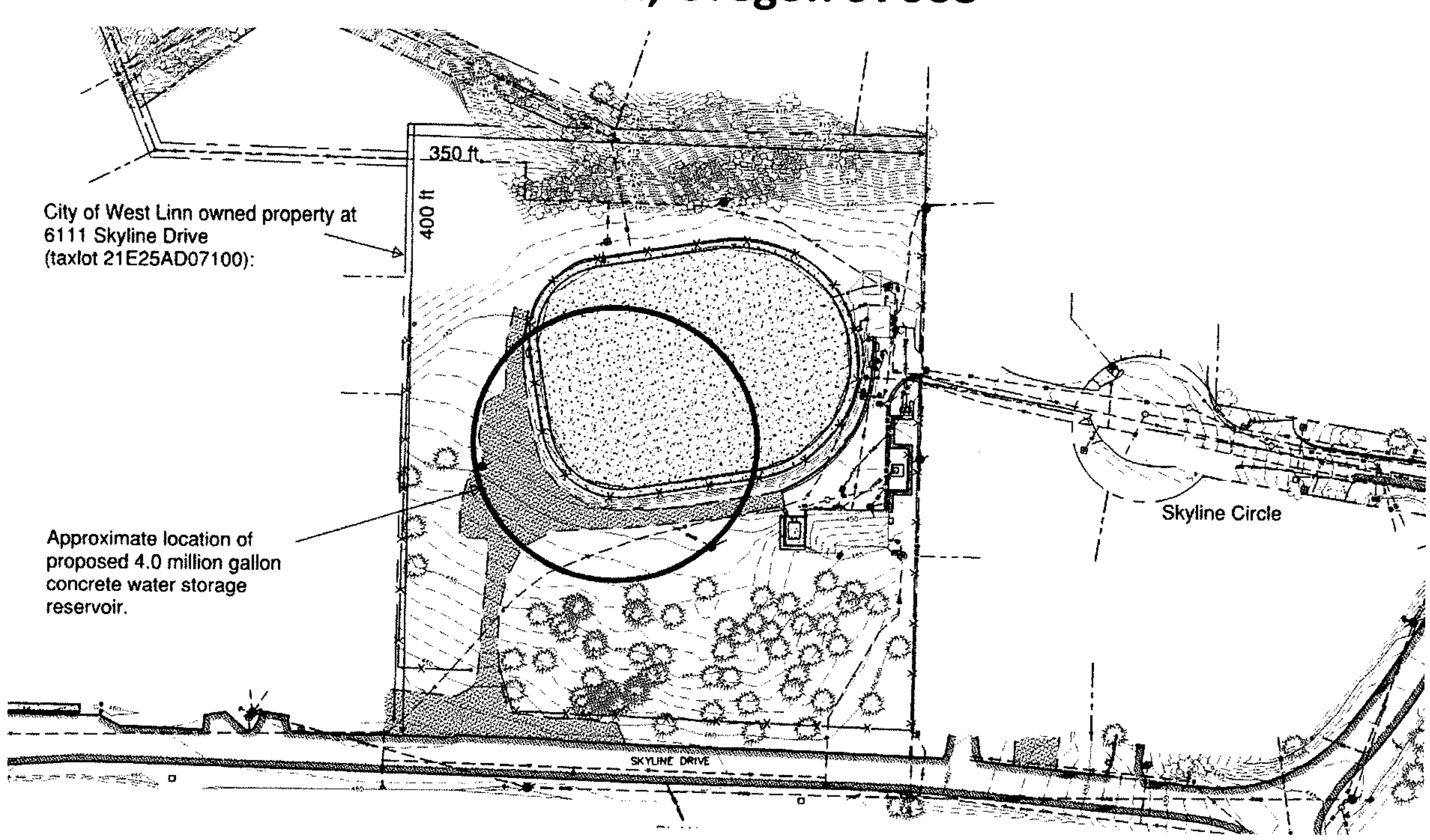
You are invited to attend a Neighborhood Meeting to discuss the water reservoir project in detail. The project will be presented at a special meeting for the Sunset Neighborhood Association, along with the Bolton Neighborhood Association and Hidden Springs/Rosemont Summit Neighborhood Association. The applicant for this project is the City of West Linn and additional information may be obtained by calling the West Linn Department of Engineering at 503-722-5500 or by email at lcalvert@westlinnoregon.gov. The meeting time and location are:

7 p.m. Thursday, April 23, 2015

West Linn Public Library

1595 Burns Street

West Linn, Oregon 97068



NEIGHBORHOOD MEETING

AFFIDAVIT OF POSTING NOTICE

CITY OF WEST LINN)

SS

CITY OF WEST LINN)

I, Jim Whynot, being duly sworn, state that I represent the party initiating interest in a proposed water reservoir affecting the land located at 6111 Skyline Drive in West Linn, Oregon, and that pursuant to Community Development Code Section 99, did on April 1, 2015, personally post notice indicating that the site may be proposed for a land use application.

A sign was posted on the fence facing Skyline Drive.

The 1st day of April, 2015.

Signature

Subscribed and sworn to, or affirmed, before me this 19th day of May, 2015.

OFFICIAL SEAL
NANCY ANN EVETT
NOTARY PUBLIC - OREGON
COMMISSION NO. 465429
MY COMMISSION EXPIRES JANUARY 31, 2016

Notary Public for the State of Oregon

County of Clackamas

My Commission Expires: January 31, 2016

Bolton Reservoir Project Neighborhood Meeting **April 23, 2015**
In accordance with Conditional Use Type 2 Process

Lance Calvert (Public Works Director/City Engineer):

- Provided background information on the project, site access, landscaping and history.
- Focus of presentation is basic understanding of land use and discussing myths and facts about the project.
- Meeting is a requirement of the CDC, Conditional Use Type 2 process. CDC is available online if citizens are interested in reviewing in more detail. CDC I s referred to during any City project and so is private development. A similar process was done during the Library Parking Lot project of 2013.
- Land Use process is about the permanent infrastructure, impact and use of land. This includes site access, lighting, landscaping and so forth. The purpose of the meeting is to get neighborhood feedback on what would be preferred. What the land use process is not is temporary things such as noise, construction, dust, traffic control, etc. However, the Municipal Code and PW Standards address these and developers (City and/or private) are required to adhere to these.
- Requirements of Land Use process includes recording of the meeting which citizens were informed of.
- Lance provided contact information to group (including for Water Dept. Supervisor Jim Whynot) and stated he was available for onsite meetings if requested.
- Final packet for Land Use application (including feedback from the meeting) will be submitted to Planning staff and will be reviewed for compliance with CDC. After which will be put on an agenda for the Planning Commission to review and make a decision on the project.

Q: With codes available for State, City, and County do you need to abide by Federal Requirements as well?
A: Local codes are typically built upon State and Federal requirements, but there are additional Federal requirements such as OSHA and BOLI, and the City follows them as well. Staff in Public Works takes jobs very seriously, and protecting public health and welfare is job 1. The City additionally consults with agencies that specialize in these types of projects such as Murray, Smith & Associates to ensure that projects are completed in compliance with regulations and safety standards.

Q: Does this process only cover the reservoir or does it include water lines as well?
A: The land use process does not include water lines. That would be a separate process, water lines is a separate entity. This includes the proposed sidewalk along Skyline's frontage of Wilderness Park. If there are questions about these projects Lance can address them after the meeting.

Q: Why now and why this particular spot. Are there additional locations where this might go and have they been thoroughly evaluated?
A: Tom Boland will discuss this during his presentation. If there are additional questions after they will be addressed.

Q: It is an old reservoir but what are the real facts about it? Why can't it just be added on to?
A: This is another point that Tom will discuss, so it may be a good point to transition to Tom's presentation of information.

Q: What is on the table for this discussion so that we don't stray from issues related to the Land Use discussion tonight?
A: Focused on the existing site and the replacement of the old reservoir within the site. For tonight, we are discussing things like, grading, site layout and design, utilities on the site, visual screening, site access, etc. When the CDC discusses things like noise, dust, odor and such, it is directed more towards the operation (permanent results of the building) not the initial construction of the facility. Prior to this meeting there were open houses on site and individual discussions with neighbors. The City has acquired two properties next door for the purposes of staging to minimize truck trips and securing equipment in the right of way.

Q: Is 6111 Skyline drive the address of the reservoir?
A: Yes

Q: Is truck traffic up and down Skyline Circle off the table for discussion?
A: That will be addressed by Tom including how we will access the site.

Project Discussion

Tom Boland (Murray, Smith and Associates):

- Purpose of tonight is to receive citizen's interest and concern to assist in finishing up the design and land use application
- The existing reservoir is a concrete slab on grade structure constructed in 1915. This means it is concrete cast with an open pit. In the early to mid-1990s a liner and cover were added to protect water quality and keep the structure tight. In the 1990's the Bolton Pump station was constructed to pump water up to the higher elevation customers in the Horton pressure zone.
- The structure has served as the hub of the City's water system for 100 years. There are now safety issues to be addressed. Primary concern is that it does not meet current seismic codes. There were no codes in 1915. Based on recent analysis, the existing structure as it stands is not stable should there be a significant earthquake. There is spalling and localized cracking especially on the north side. The liner has had major repairs in 2008 and 2012. The last 3 Water Master Plans have called for the replacement of this reservoir dating back to 1999.
- 1st step going back to last year was a siting analysis which looked at potential alternative sites which requires the structure to be at a particular elevation to proper water pressure for residents can be achieved. All properties within the required band of elevation were examined. There are not many sites available that are within appropriate elevation.

Q: Did the previous master plans call for a 4mg tank?
A: Current and adopted master plan calls for 4mg tank. Lance provided additional information regarding West Linn's varying elevations and storage of water. If storage is too high or too low, the City is paying additional electricity to pump the water to the various elevations to meet demand. The City has 16 pressure zones which is a lot for a city of this size. The City has to be conscious of water pressure, it can't be too low (trickling out of the faucet) or too high. So when total storage is discussed, standards are based on needs for fire flow, operating range (storing water at night during low use so that high levels of use can be accommodated during the day), Emergency fire demands, if water service goes off line. If South Fork distribution line is blocked (where City gets its water), there is a need for reservoirs to have back up storage to continue water system. States have requirements for total water storage, the State of Oregon does not require Cities to meet a certain threshold but the City of West Linn has 5.5 mg of storage system wide which is lower than Washington's mandated threshold for a city of similar size. The bigger the storage, the more cost effective the storage is. Having additional storage also assists in deficits in the Willamette River and Robinwood pressure zones.

Q: Are there current deficits in the water system?
A: Yes, fire-flow and worse case scenarios would provide a deficit in these areas.

Q: Alice Richmond stated that Washington County is having a drought and there is discussion of taking water from Willamette through Tigard. Will that impact our water levels?
A: Tualatin Valley Water District is looking at a program to tap into the Willamette to get off the Portland Water system. Short answer, is that this will not have direct impact on the City of West Linn's water system or supply.

- Looking at the band of elevation only 16 properties had adequate topography and size to accommodate the reservoir. Examined proximity to existing pump station, conflicts with natural environments such as streams and WRA, current area use and impact to existing residences. Through that 3 sites were deemed viable including the existing and 2 sites in Wilderness Park. The advantages of the existing site (preferred site) is a large flat area, close proximity to existing pump station and key water lines in the area (not changing these required existing infrastructures keeps overall costs down).
- Next step was to complete geotechnical investigation and geologic analysis to ensure that it was safe and geotechnical suitable for the work. Included soil borings, slope stability analysis, evaluation of the large ancient landslide area, site specific seismic hazard study.

Q: Was boring completed at the 2 other alternative sites?
A: No, but the existing site has always been the preferred site, so when we looked at other locations we looked to see if any of them made more sense. After extensive review, it was determined that the existing site is still preferred, so rather than go through another round of analysis of sites which would be more expensive to build on, the City chose not to spend additional funds on boring, slope stability analysis, etc. at the additional 2 sites.

Q: If boring wasn't done at the other locations, how can you really compare and state that one is better than the other? Cost analysis isn't primary concern of mine, safety and long range planning is. If there isn't comparable geological information available for all three sites, how can you state which site is most safe?
A: The City always puts safety of citizen's first. Experts that have reviewed this information and are the ones determining safety of the areas include nationally renowned experts in slides and PhD qualified geologists. In addition to consultants brought on board, the City has had independent expert professor at Portland State University (Scott Burns) engaged in the process and everything recommended by him has been included in the analysis.

Q: Are the underground springs that exist in the hillside addressed through the geotechnical investigations or any of the other studies that were completed?
A: The slope stability model includes the spring underwater system of the hillside to analyze stability.

- What was found through the slope stability model is that the existing structure is unstable and some issues that exist beyond the physical structure include a large amount of fill dirt that was brought in during the initial construction in 1915. The proposed project includes moving the reservoir back from the slope as recommended through the slope stability analysis, reducing and softening the slope of the grade so it is less steep, address foundation drainage and add geo-piers (aggregate piers that strengthen the soil beneath the slab of the tank). In addition, the project proposes removing

6,000 tons of soil from the site (equivalent to 500 school buses), the safety of the site and the reservoir structure will be greatly improved.

- Another critical safety issue is the structure itself. The new tank will be pre-stressed, concrete, cast in place reservoir in accordance with AWWA standards. This design was specifically developed for the earthquake prone west coast. It has performed extremely well in the last 60 years in the largest seismic events throughout the West Coast. The structure itself will be a huge safety improvement

Q: What is pre-stressed mean?
A: Concrete walls are post-tension and walls are wrapped with 7 strand seismic cable. Concrete is strongest with compression, so squeezing and pre-stressing concrete then filling with water allows for compression on both sides and creates the strongest type of loading that a concrete structure can bare.

Q: How does this proposed reservoir compare with the reservoir that was recently put in Portland and leaked?
A: That was a conventional reinforced concrete tank, relaying on massive walls and heavy rebar. This is not the same type of structure.

Q: Does the current reservoir leak?
A: It did previously, but a liner was previously installed. Current liner has a 20 year service life. The floating cover has a 20 year service life as well. There has been tears in the liner which have been fixed but there is possible of leakage from liner should it fail.

Q: How do we know this next reservoir won't leak?
A: Many reservoirs of this type have been built in the region and they have not leaked. A rubber membrane is installed beneath the foundation and a radial draining system is installed which would inform staff if there was a leak. Should there be some sort of leak through the floor it would be detected. This also allows the ability to see the difference between ground water coming from below and water leaking from the reservoir.

- Covered water is now a requirement of reservoirs to keep water safe and protect the water from outside contaminants. New reservoir will have a concrete top which will make it much safer than the current liner which has the possibility of tearing. There are additional concerns about the liner on the current reservoir as staff at times would have to walk on it to repair and like a pool cover; it does have the possibility of tearing and allowing someone to fall through into the water. There are fences and barbwire to keep people out of the site but a new tank would eliminate the possibility of someone falling through the liner.

Q: How would the proposed concrete cover perform in an earthquake?
A: There is flexibility between the floor joints, roof, floor, and wall. There are bearing pads and shear cans so as there is movement the cover floats a little bit.

Q: What is the current capacity of the reservoir and what would the capacity be of the proposed new reservoir? What will it look like?
A: Current reservoir is 2.5MG and proposed reservoir will be 4.0MG

- From the street side the new reservoir will have 5' wall exposed, and there will be 15' of exposed wall on the downslope side of the reservoir. There will be Stormwater detention and water quality facilities as required on site. There will be some removal of trees around the existing house which will be demolished. The goal is to preserve as much perimeter landscaping as possible. The new tank will actually have a smaller surface area wise due to the change in the shape of the reservoir.

Q: What is the total depth of the new reservoir from surface of water down?
A: 25'

Q: What becomes of the staging area after construction?
A: Council approved the purchase of the staging area under condition that the properties are sold after construction is completed and funds will be returned to the water fund.

Q: Would additional weight from the reservoir have impact on the site and land below the reservoir site?
A: No, net loading will actually be less by about 6,000lbs due to removal of soil (soil weighs more than water).

- Basalt is beneath the soil and is extremely tough, there are multiple geo-piers to help with stability and work is fairly simple to complete. There will not be a need to bore into Basalt in order to install piers.

Q: There are properties adjacent to staging areas. Will there be a lot of change to landscape that would impact these properties?
A: New homes would likely be built on these lots after construction and would be subject to Land Use applications.

- Feedback regarding preserving trees in area is good information to be provided to the City. This is an important issue to residents in the area. Having large trees is a desirable part of living in this area.

Q: What is the timeline for construction?
A: Reservoir could be taken off line and demo could start in spring of 2016. The goal is to complete reservoir construction is summer 2017. Total timeline is 18 months with 5 day work weeks.

Q: What will be the impact of water service during this time?
A: LOT treatment plant intertie will be used to shave off peaks usage during the summer. There will be temporary connections between the Horton and Bolton zones and the City doesn't anticipate any disruption to water service. LOT will be making water by this time.

- There are other ways to move water around when needed. The Rosemont Reservoir is currently off service while safety improvements are completed and residents in that area are getting water from other pressure zones. The morning is the most challenging for water service as this is the highest amount of water usage in the City during a single time period of the day.

Q: Will the new reservoir be fenced off or will you be able to walk around it?
A: Currently there is a chain link fence with barbed wire. With the new more secured reservoir there will be a 6ft black coated fence and in the front a decorative iron fence and gate is planned. There is no plan for barbed wire. For planting, we plan to get rid of invasive species, and plant new screening. The current pump station has a flat roof which is a maintenance issue. There are plans to improve the aesthetics of the pump station and have a pitched roof.

Q: What powers the pump station?
A: Electricity and there is a back-up generator onsite that is fueled by an onsite diesel tank.

Q: Where does Stormwater runoff go?
A: Stormwater currently runs to a storm drain and it will continue to do so. New impervious surfaces will have new Stormwater quality facilities to receive runoff which are standard code requirements now. The detention facility will be lined so none of the water will go into the rock face and will have a leak detection system.

Q: Where is Bolton Creek?
A: Tom provided visual information on the location of this creek.

Q: Will the fence complete close the site?
A: The goal is to fence the perimeter of the site.

Q: Is the water in the reservoir treated?
A: The water is treated prior to coming to the reservoir and it is not chlorinated within the reservoir. Water quality will be improved with the new construction. Separate inlet and outlet control will allow for cycling of the water more easily and keep it fresh.

Q: Where does treatment occur?
A: South Fork Water Treatment Plant treats all the water prior to distributing to West Linn.

Q: Is Skyline Drive stable? Is it a stable area for staging during construction? What about all the heavy loads driving on the street. Can we incorporate improvements to the road?
A: Skyline Dr. isn't part of the process but there is room for improvement on the street. The City is currently looking at improvements on Skyline Dr. including the sidewalk and improvements to the slope along Skyline. The staging area will minimize the loads on the streets. Skyline does not currently have any truck load limits. Trucks do have load limits though and if concerns occur we can have them reduce their loads.

Q: Is concrete going to be brought in in ready mix trucks?
A: Approximately 300 truck-loads over the span of approximately 8 months.

Q: What will the final product look like with regards to plants? There are lots of blackberries on property.
A: The fence will not be a solid wall so that you can still see the greenery of the area. The street side we want to maintain visibility of the site for safety. The City wishes to install native vegetation near the adjacent properties to minimize mowing and other maintenance issues. If there are specific requests for planting please let the City know so they can consider. When trees are young they will provide screening and as they grow the shrubs that will be planted will be developed to provide screening as well. A landscape architect plan will be put in place.

Q: There will be a lot of soil coming off the site. Can there be a neighborhood plan that indicates preferred routes that the citizen would like contractors to use?
A: This will be planned during construction and the City can incorporate truck routes into bid specifications. It can be very difficult to enforce these routes though because Police cannot always determine which trucks are related to which project.

Q: Who would citizen's need to contact about truck routes?

A: City staff including Public Works Director or Assistant City Engineer/Project Manager. You can also contact Morgan (Director's assistant) to schedule a time to meet with City staff. Contact information was provided. Please call early if issues arise. It is much easier to address during the issue rather than try to fix after the fact.

Q: How bad will noise levels be?
A: It is expected to be similar to a home being built next door. During demolition there may be concrete saws, excavator noise, etc. The goal is to minimize noise issues but with all construction there will be some noise. During construction of the new reservoir there will be concrete pouring mostly which may be quieter than the traditional building of a home.

Q: Will the new site require heavy regular maintenance?
A: It will be less maintenance than it is now, most maintenance will be mowing of the property and exercising of generators. Exercising of generators is not noisy and has been done for years without neighbor complaints.

- Russ Axelrod expressed concerns about nuances of the hydrology in the area.
- Lance and Russ are attempting to speak to Dr. Burns and GRI to identify any remaining concerns.

Q: Will vibrations from the aggregate piers installation cause issues?
A: The most cost effective approach is to use auger style installation and that would likely be the method. This is viewed as a safer approach and will minimize vibration during installation.

Q: Does this structure have a set back from the road required? Can it be set back more?
A: There is a set-back requirement and the City is adhering to that. The City can look at moving it more and can speak to GRI about if this is a cost-effective possibility. Doing this would require removal of additional trees and vegetation and the City wants to be mindful of this as well.

Q: Should a 3MG reservoir be sufficient? This is what was called for in the 2008 Master Plan and would (according to 2008 Master Plan) cover peak hour demands.
A: The Bolton zone is the hub of the City's water system. The 3MG capacity would cover peak usage for the Bolton area but the Rosemont area's usage is higher. The City is taking in consideration the entire water systems demand for water to ensure that there is adequate water to feed into other pressure zones if needed. The Bolton Reservoir needs to compensate for the smaller Rosemont Reservoir. The Rosemont Reservoir is the smallest reservoir and serves some of the highest water usage.

Q: What is the future water system plans for the Rosemont area since the water usage is so high and there continues to be development in the area?
A: Infrastructure to deal with water demands are now built out in the area including a new pump station was completed in the Bland pressure zone.

- Every 8-10 years a Master Plan is updated, Councilor Perry has been on a tour of the water system and is becoming more acquainted with the system and citizen are welcome to do so as well to become better educated about our City's long term water needs.

Q: Is there such a thing as too much capacity? Why not 5MG?
A: The City relays on analysis of experts to review the City's actual water needs. Utilizing interties more emergencies reduces the need for more capacity in the reservoir. Without interties we would need

8MG capacity reservoir. 4MG will handle the maximum possibly built out for the City of West Linn (including future development of vacant land and infill). That is what the system is designed to support.

- If there are additional questions City staff will be available.

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May 14, 2015

Planning and Building
City of West Linn
22500 Salamo Road #1000
West Linn, Oregon 97068

Re: Arborist Report and Tree Preservation Plan for Bolton Reservoir
West Linn, Oregon
Project No. MHA15032 Bolton Reservoir

Please find enclosed the Arborist Report and Tree Preservation Plan for the Bolton Reservoir replacement project located at 6111 Skyline Drive in West Linn, Oregon. Please contact us if you have questions or need any additional information.

Respectfully,
Morgan Holen & Associates, LLC

Morgan E. Holen

Morgan E. Holen, Owner
ISA Certified Arborist, PN-6145A
ISA Tree Risk Assessment Qualified
Forest Biologist

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Arborist Report and Tree Preservation Plan

Bolton Reservoir
West Linn, Oregon

May 14, 2015

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Bolton Reservoir – West Linn, Oregon
Arborist Report and Tree Preservation Plan
May 14, 2015

MHA15032

Purpose

This Arborist Report and Tree Preservation Plan for the Bolton Reservoir replacement project in West Linn, Oregon, is provided pursuant to City of West Linn Community Development Code Chapter 55, Municipal Code Sections 8.500 and 8.600, and the West Linn Tree Technical Manual. This report describes the existing trees located on and directly adjacent to the project site, as well as recommendations for tree removal, retention and protection. This report is based on observations made by International Society of Arboriculture (ISA) Certified Arborist (PN-6145A) and Qualified Tree Risk Assessor Morgan Holen during a site visit conducted on May 8, 2015 (which included a site meeting with the City Arborist Mike Perkins) and site plan coordination with Murray, Smith & Associates.

Scope of Work and Limitations

Morgan Holen & Associates was contracted by Murray, Smith & Associates to collect tree inventory data for individual trees measuring six inches and larger in diameter and to develop an arborist report and tree preservation plan for the project. The project will replace the existing 100 year old 2.5 million gallon Bolton Reservoir with a new 4.0 million gallon water reservoir. Site plans were provided by Murray, Smith & Associates illustrating the location of existing trees and potential construction impacts.

Visual Tree Assessment (VTA) was performed on individual trees located across the site; a few trees located off-site to the north directly adjacent to the project site were also surveyed and inventoried. VTA is the standard process whereby the inspector visually assesses the tree from a distance and up close, looking for defect symptoms and evaluating overall condition and vitality of individual trees. Trees were evaluated in terms of general condition and potential construction impacts. Following the inventory fieldwork, we coordinated with Murray, Smith & Associates to discuss tree protection recommendations.

The client may choose to accept or disregard the recommendations contained herein, or seek additional advice. Neither this author nor Morgan Holen & Associates, LLC, have assumed any responsibility for liability associated with the trees on or adjacent to this site.

General Description

The Bolton Reservoir site includes a water reservoir and pump station with existing trees located primarily north and south of the reservoir.

A dense stand of bigleaf maples (*Acer macrophyllum*) is located along the northern property boundary on an old fill slope. These trees are heavily infested with invasive English ivy (*Hedera helix*), which is growing up the trunks and into the crowns of many trees. While the stand appears in fair condition as an undisturbed intact group at this time, the ivy can be expected to overtop and kill trees. In addition, many of these trees have multiple leaders, trunk decay, and small high live crowns.

Just beyond the northern property boundary, a 25-foot wide stormwater easement is located to the northwest, heading downslope towards Bolton Creek. Several trees are located within the easement, including four bigleaf maples, one Douglas-fir (*Pseudotsuga menziesii*) and two grand firs (*Abies grandis*) which all appear in relatively good condition.

South of the reservoir and adjacent to Skyline Drive, the existing trees are comprised of stand grown even-aged Douglas-fir. The individual trees are variable in condition, but the group as a whole is considered to be in good condition. Many of these trees have old broken tops with new leaders, some of which appear better adapted and more stable than others. The group is undergoing natural stand dynamics and dominant trees are outcompeting and suppressing less vigorous trees. Douglas-fir is not tolerant of shade, so trees growing in close proximity to one another have been competing with and adapting to one another over time. Trees located in the interior of groups are more suppressed because they do not receive as much sunlight. Edge grown trees generally have the longest and densest live crowns, but the crowns are mainly one-sided to the exterior of the stand. The trees located in this stand are most sustainable and suitable for retention intact, as an undisturbed group in their relatively natural condition; removal of individual trees from the group will likely expose remaining trees making them more susceptible to windthrow and potentially hazardous. Individual stand grown trees were evaluated in terms of potential impacts from adjacent tree removal.

Tree Inventory

In all, 151 existing trees were inventoried, including seven different tree species. Nine of the inventoried trees are located off-site on neighboring properties. Trees located within the easement area are considered on-site for the purposes of this analysis. Table 1 provides a summary of the number of inventoried trees by species and location. The enclosed tree data provides a complete description of the individual trees; site plan drawings illustrate the location of trees by corresponding point number.

Table 1. Number of Inventoried Trees by Species and Location – Bolton Reservoir.

Common Name	Species Name	Off-site	On-site	Quantity	Percent
bigleaf maple	*Acer macrophyllum*	8	73	81	54%
Douglas-fir	*Pseudotsuga menziesii*	1	63	64	42%
grand fir	*Abies grandis*	0	2	2	1%
spruce	*Picea* spp.	0	1	1	<1%
western redcedar	*Thuja plicata*	0	1	1	<1%
white fir	*Abies concolor*	0	1	1	<1%
white pine	*Pinus monticola*	0	1	1	<1%
Total		**9**	**142**	**151**	**100%**

Bigleaf maple and Douglas-fir account for 96-percent of the inventoried trees. These trees are primarily located in dense stands north and south of the existing reservoir as previously described. A few individual Douglas-firs are also located outside of the southern stand, including two near the northwest corner of the site and one within the easement area. Eight bigleaf maples and one Douglas-fir are located off-site to the north. The remaining trees include a mix of species in variable condition. This includes two large grand firs in good condition in the easement area, and four small evergreens in fair and poor condition that appear to have been planted near the western property boundary, west of the existing reservoir—a spruce (*Picea* spp.), western redcedar (*Thuja plicata*), white fir (*Abies concolor*), and white pine (*Pinus monticola*).

Significant trees will be determined by the City Arborist. Based on our evaluation of the size, type, location, health, and long term survivability of the individual trees located on site, 56 of the 142 on-site trees were identified as potentially being significant.

Tree Preservation Plan

We coordinated with the project team to discuss trees suitable for preservation in terms of potential construction impacts and site plan alternatives. Proposed tree removal is mainly for the purposes of construction, including grading, construction of a new water reservoir, construction of a new access road, and stormwater line installation on a steep slope; however, a few trees in the Douglas-fir stand are recommended for removal because of condition, including dead, dying, and high risk hazard trees. Trees located off-site on neighboring properties will be protected during construction. Table 2 provides a summary of the number of non-significant and potentially significant on-site trees by treatment recommendation.

Table 2. Number of On Site Trees by Treatment Recommendation and Significance.

Treatment	Remove	Retain	Total
Non-Significant Trees	81	5	**86**
Potentially Significant Trees	28	28	**56**
Total	**109**	**33**	**142**

Of the 142 on-site trees, 109 are recommended for removal either for construction or because of poor or hazardous condition, including 28 potentially significant trees. The remaining 33 trees are recommended for retention, including 28 potentially significant trees which are all stand grown Douglas-firs located south of the existing reservoir.

These 28 trees are good candidates for preservation and were each assessed in terms of suitability for preservation with adjacent tree removal. Nevertheless, the trees planned for preservation should be re-evaluated at the time of clearing in terms of exposure from adjacent tree removal and the potential for increased risk. At the time of clearing, a qualified arborist should assess the remaining trees in terms of general condition, exposure from adjacent tree removal, height to diameter ratio, live crown ratio, and overall hazard risk potential. If trees are determined to be high risk, the arborist should document their findings and recommendations in a report for the City; the trees planned for preservation should continue to be protected until if and when the City provides written authorization for their removal.

Recommendations for tree protection are provided in the next section.

Tree Protection Standards

Trees to be protected will need special consideration to assure their protection during construction. It is the Client's responsibility to implement this plan and to monitor the construction process. The project arborist will be available during construction to help with tree related issues as needed. Tree protection measures include:

Before Construction

1. **Preconstruction Conference.** The project arborist should be on site to discuss methods of tree removal and tree protection prior to any construction.
2. **Tree Protection Zone.** The project arborist should designate the Tree Protection Zone (TPZ) for each tree to be protected. Where feasible, the size of the TPZ should be established at the dripline of the tree plus 10-feet. Alternatively, the TPZ should be established at the dripline of protected trees. Where infrastructure must be installed closer to the tree(s), the TPZ may be established within the dripline area if the project arborist, in coordination with the City Arborist, determines that the tree(s) will not be unduly damaged. The location of TPZs should be shown on construction drawings.
3. **Protection Fencing.** Protection fencing should serve as the tree protection zone and should be erected before demolition, grubbing, grading, or construction begins. All trees to be retained should be protected by six-foot-high chain link fences installed at the edge of the TPZ. Protection fencing should be secured to two-inch diameter galvanized iron posts, driven to a depth of a least two feet, placed no further than 10-feet apart. If fencing is located on pavement, posts may be supported by an appropriate grade level concrete base. Protection fencing should remain in place until final inspection of the project permit, or in consultation with the project arborist.
4. **Signage.** An 8.5x11 –inch sign stating, "WARNING: Tree Protection Zone," should be displayed on each protection fence at all times.
5. **Designation of Cut Trees.** Trees to be removed should be clearly marked with construction flagging, tree-marking paint, or other methods approved in advanced by the project arborist. Trees should be carefully removed so as to avoid either above or below ground damage to those trees to be preserved. Roots of stumps that are adjacent to retained trees should be carefully severed prior to stump extraction.
6. **Hazard Tree Assessment.** At the time of clearing, the project arborist should re-evaluate trees planned for preservation in terms of general condition, exposure from adjacent tree removal, and overall hazard risk potential. High risk trees, if any, should be documented in a written report; trees should continue to be protected until the City authorizes their removal.
7. **Verification of Tree Protection Measures.** Prior to commencement of construction, the project arborist should verify in writing to the City Arborist that tree protection fencing has been satisfactorily installed.

During Construction

8. **Tree Protection Zone Maintenance.** The protection fencing should not be moved, removed, or entered by equipment except under direction of the project arborist, in coordination with the City Arborist.
9. **Storage of Material or Equipment.** The contractor should not store materials or equipment within the TPZ.

10. **Excavation within the TPZ.** Excavation with the TPZ should be avoided if alternatives are available. If excavation within the TPZ is unavoidable, the project arborist should evaluate the proposed excavation to determine methods to minimize impacts to trees. This can include tunneling, hand digging or other approaches. All construction within the TPZ should be under the on-site technical supervision of the project arborist, in coordination with the City Arborist.

11. **Tree Protection Zone.** The project arborist should monitor construction activities and progress, and provide written reports to the developer and the City at regular intervals. Tree protection inspections should occur monthly or more frequently if needed.

12. **Quality Assurance.** The project arborist should supervise proper execution of this plan during construction activities that could encroach on retained trees. Tree protection site inspection monitoring reports should be provided to the Client and City on a regular basis throughout construction.

Post Construction

13. **Final Report.** After the project has been completed, the project arborist should provide a final report to the developer and the City. The final report should include concerns about any trees negatively impacted during construction, and describe the measures needed to maintain and protect the remaining trees for a minimum of two years after project completion.

Please contact us if you have questions or need any additional information. Thank you for choosing Morgan Holen & Associates, LLC, to provide consulting arborist services for the Bolton Reservoir replacement project.

Thank you,
Morgan Holen & Associates, LLC

Morgan E. Holen

Morgan E. Holen, Owner
ISA Certified Arborist, PN-6145A
ISA Tree Risk Assessment Qualified
Forest Biologist

Enclosures: MHA15032 Bolton Reservoir – Tree Data 5-8-15

No.	Common Name	Species Name	DBH*	C-Rad^	Cond#	Defects and Comments	Sig?	Treatment
1000	Douglas-fir	*Pseudotsuga menziesii*	41	28	G	codominant crown class, some asymmetry	Yes	remove
1001	Douglas-fir	*Pseudotsuga menziesii*	47	35	G	codominant crown class, natural lean to south, some asymmetry	Yes	remove
1002	Douglas-fir	*Pseudotsuga menziesii*	28	12	F	codominant crown class, lower trunk swelling, abnormal bark, suspect basal and trunk decay	No	remove
1003	Douglas-fir	*Pseudotsuga menziesii*	26	20	G	edge of stand, very one-sided to north	Yes	remove
1004	Douglas-fir	*Pseudotsuga menziesii*	29	14	G	codominant crown class, one *Phellinus pini* conk observed on lower trunk	Yes	remove
1005	Douglas-fir	*Pseudotsuga menziesii*	2x10	10	F	intermediate crown class, codominant stems ~1' above ground level	No	remove
1006	Douglas-fir	*Pseudotsuga menziesii*	22	22	F	codominant crown class, very one-sided crown	Yes	remove
1007	Douglas-fir	*Pseudotsuga menziesii*	22	14	F	intermediate crown class, suppressed below 1006 and 1008	Yes	remove
1008	Douglas-fir	*Pseudotsuga menziesii*	32	26	G	codominant crown class, crown asymmetry	Yes	remove
1009	Douglas-fir	*Pseudotsuga menziesii*	26	12	F	old broken top, forked off-center leaders, all live crown above this juncture, old trunk wounds ~0-20' on both SE and W faces, increased risk with adjacent tree removal	Yes	remove
1010	Douglas-fir	*Pseudotsuga menziesii*	26	13	F	old broken top, one new leader, all live crown above this juncture, increased risk with adjacent tree removal	Yes	remove
1011	Douglas-fir	*Pseudotsuga menziesii*	22	10	G	old broken top, one new leader, all live crown above this juncture, re-evaluate at clearing	Yes	retain
1012	Douglas-fir	*Pseudotsuga menziesii*	18	12	F	old broken top, new leader with moderate structure, one-sided crown to south, re-evaluate at clearing	Yes	retain

No.	Common Name	Species Name	DBH*	C-Rad^	Cond#	Defects and Comments	Sig?	Treatment
1013	Douglas-fir	*Pseudotsuga menziesii*	2x20	12	F	codominant stems at ~4', some included bark, old broken top with new leader, one-sided crown to south, re-evaluate at clearing	Yes	retain
1014	Douglas-fir	*Pseudotsuga menziesii*	15	10	P	old basal wound, old broken top, suppressed beneath 1013, one-sided crown to street, increased risk	No	remove
1015	Douglas-fir	*Pseudotsuga menziesii*	12	6	P	suppressed, severe *Phellinus pini* infection, high risk to street	No	remove
1016	Douglas-fir	*Pseudotsuga menziesii*	14	6	P	suppressed, broken top, forked leaders, mostly dead	No	remove
1017	Douglas-fir	*Pseudotsuga menziesii*	8	0	D	dead	No	remove
1018	Douglas-fir	*Pseudotsuga menziesii*	28	12	F	codominant crown class, crown asymmetry, re-evaluate at clearing	Yes	retain
1019	Douglas-fir	*Pseudotsuga menziesii*	24	12	F	codominant crown class, crown asymmetry, re-evaluate at clearing	Yes	retain
1021	Douglas-fir	*Pseudotsuga menziesii*	24	14	F	codominant crown class, crown asymmetry, sweep in main stem, re-evaluate at clearing	Yes	retain
1022	Douglas-fir	*Pseudotsuga menziesii*	6	4	P	suppressed, not viable	No	remove
1023	Douglas-fir	*Pseudotsuga menziesii*	6	4	P	suppressed, not viable	No	remove
1024	Douglas-fir	*Pseudotsuga menziesii*	26	18	F	codominant crown class, crown asymmetry, retain with 1025	Yes	retain
1025	Douglas-fir	*Pseudotsuga menziesii*	34	16	G	old broken top, new leader appears well adapted, re-evaluate at clearing	Yes	retain
1026	Douglas-fir	*Pseudotsuga menziesii*	6	6	P	suppressed, mostly dead	No	remove
1027	Douglas-fir	*Pseudotsuga menziesii*	16	12	F	intermediate crown class, crown asymmetry	Yes	remove
1028	Douglas-fir	*Pseudotsuga menziesii*	22	16	G	codominant crown class, crown asymmetry	Yes	remove

No.	Common Name	Species Name	DBH*	C-Rad^	Cond#	Defects and Comments	Sig?	Treatment
1029	Douglas-fir	*Pseudotsuga menziesii*	43	36	G	codominant crown class, edge of stand, minor asymmetry	Yes	remove
1030	Douglas-fir	*Pseudotsuga menziesii*	30	16	P	trunk wound ~0-20' W face, resin flow, large witches broom in crown	No	remove
1031	Douglas-fir	*Pseudotsuga menziesii*	36	26	G	old broken top, new leader appears well adapted	Yes	remove
1032	Douglas-fir	*Pseudotsuga menziesii*	30	16	G	dominant crown class, no major defects	Yes	remove
1033	Douglas-fir	*Pseudotsuga menziesii*	17	10	F	intermediate crown class, only suitable for retention with tree 1032	Yes	remove
1034	Douglas-fir	*Pseudotsuga menziesii*	36	16	G	codominant crown class, crown asymmetry, old broken top, few dead branches, re-evaluate at clearing	Yes	retain
1035	Douglas-fir	*Pseudotsuga menziesii*	30	12	F	codominant crown class, one-sided crown, re-evaluate at clearing	Yes	retain
1036	Douglas-fir	*Pseudotsuga menziesii*	46	20	G	codominant crown class, lower trunk swelling, leaders appears to originate from old top at ~10', remove ivy at base and lower trunk	Yes	retain
1037	Douglas-fir	*Pseudotsuga menziesii*	8	6	P	suppressed, not viable	No	remove
1038	Douglas-fir	*Pseudotsuga menziesii*	34	20	F	codominant crown class, crown asymmetry	Yes	retain
1039	Douglas-fir	*Pseudotsuga menziesii*	34	14	F	codominant crown class, forked leaders	Yes	retain
1040	Douglas-fir	*Pseudotsuga menziesii*	32	22	G	codominant crown class, some asymmetry	Yes	retain
1041	Douglas-fir	*Pseudotsuga menziesii*	48	26	G	dominant crown class, few dead and broken branches	Yes	retain
1042	Douglas-fir	*Pseudotsuga menziesii*	42	20	F	old broken top, lightening scar, dead branches, moderate structure, some increased risk potential to power lines and adjacent home, safety pruning/aerial inspection recommended	Yes	retain
1043	Douglas-fir	*Pseudotsuga menziesii*	22	14	F	broken top, small new leader	Yes	retain

No.	Common Name	Species Name	DBH*	C-Rad^	Cond#	Defects and Comments	Sig?	Treatment
1044	Douglas-fir	*Pseudotsuga menziesii*	26	22	F	broken top, small new leader	Yes	retain
1045	Douglas-fir	*Pseudotsuga menziesii*	22	10	F	forked leaders, low target potential	Yes	retain
1046	Douglas-fir	*Pseudotsuga menziesii*	34	20	G	codominant crown class, minor asymmetry	Yes	retain
1047	Douglas-fir	*Pseudotsuga menziesii*	16	14	F	intermediate crown class, crown asymmetry, re-evaluate at clearing	Yes	retain
1048	Douglas-fir	*Pseudotsuga menziesii*	31	18	G	codominant crown class, some asymmetry, re-evaluate at clearing	Yes	retain
1049	Douglas-fir	*Pseudotsuga menziesii*	24	8	F	minor pistol-butt, old broken top, epicormic sprouts, interior of group	Yes	retain
1050	Douglas-fir	*Pseudotsuga menziesii*	26	18	F	codominant crown class, high live crown, suspicious resin flow ~0-10' N face, re-evaluate at clearing	Yes	retain
1051	Douglas-fir	*Pseudotsuga menziesii*	32	22	F	codominant crown class, one-sided crown to N, large surface roots on NNW side	Yes	remove
1052	Douglas-fir	*Pseudotsuga menziesii*	30	26	G	codominant crown class, crown asymmetry, old broken top, new leader somewhat off-center but appears well adapted	Yes	remove
1053	Douglas-fir	*Pseudotsuga menziesii*	14	6	P	suppressed, not viable	No	remove
1054	Douglas-fir	*Pseudotsuga menziesii*	30	16	F	codominant crown class, high live crown, old broken top, minor pitch flow ~0-10' N face	Yes	remove
1055	Douglas-fir	*Pseudotsuga menziesii*	32	16	F	codominant crown class, high live crown, re-evaluate at clearing	Yes	retain
1056	Douglas-fir	*Pseudotsuga menziesii*	18	12	F	intermediate crown class, only suitable for retention with tree 1032	Yes	remove
1058	white fir	*Abies concolor*	6	8	P	dead branches, trunk and branch decay	No	remove
1059	western redcedar	*Thuja plicata*	4x8	10	F	codominant stems ~1' above ground level, poor structure	No	remove

No.	Common Name	Species Name	DBH*	C-Rad^	Cond#	Defects and Comments	Sig?	Treatment
1060	spruce	*Picea* spp.	8	8	F	few dead branches	No	remove
1061	white pine	*Pinus monticola*	10	10	F	topped in the past, new leaders with poor structure	No	remove
1783	Douglas-fir	*Pseudotsuga menziesii*	36	20	G	dead branches, spur leader, no major defects	Yes	retain
1786	Douglas-fir	*Pseudotsuga menziesii*	30	20	G	codominant crown class, crown asymmetry, few dead branches overhand street, remove ivy from base and lower trunk	Yes	retain
1803	Douglas-fir	*Pseudotsuga menziesii*	30	18	G	codominant crown class, crown asymmetry, self-correcting lean, old broken top, few dead branches overhang street	Yes	retain
9653	bigleaf maple	*Acer macrophyllum*	2x6	10	F	not evaluated	Off-site	protect
9655	bigleaf maple	*Acer macrophyllum*	24	16	F	extensive ivy infestation	No	remove
9656	bigleaf maple	*Acer macrophyllum*	24	16	F	extensive ivy infestation	No	remove
9657	bigleaf maple	*Acer macrophyllum*	14	12	F	extensive ivy infestation	No	remove
9658	bigleaf maple	*Acer macrophyllum*	14	12	F	extensive ivy infestation	No	remove
9659	bigleaf maple	*Acer macrophyllum*	2x14	12	P	extensive ivy infestation	No	remove
9662	bigleaf maple	*Acer macrophyllum*	16	12	F	extensive ivy infestation	No	remove
9663	bigleaf maple	*Acer macrophyllum*	8	10	P	extensive ivy infestation	No	remove
9664	bigleaf maple	*Acer macrophyllum*	6	10	P	extensive ivy infestation	No	remove
9665	bigleaf maple	*Acer macrophyllum*	2x12	12	P	extensive ivy infestation	No	remove
9666	bigleaf maple	*Acer macrophyllum*	8	10	P	extensive ivy infestation	No	remove
9667	bigleaf maple	*Acer macrophyllum*	12	12	F	extensive ivy infestation	No	remove
9668	bigleaf maple	*Acer macrophyllum*	12	12	F	extensive ivy infestation	No	remove
9669	bigleaf maple	*Acer macrophyllum*	12	12	F	extensive ivy infestation	No	remove
9670	bigleaf maple	*Acer macrophyllum*	14	12	F	extensive ivy infestation	No	remove
9671	bigleaf maple	*Acer macrophyllum*	16	14	F	extensive ivy infestation	No	remove
9672	bigleaf maple	*Acer macrophyllum*	12	12	F	extensive ivy infestation	No	remove

No.	Common Name	Species Name	DBH*	C-Rad^	Cond#	Defects and Comments	Sig?	Treatment
9673	bigleaf maple	*Acer macrophyllum*	8	10	P	extensive ivy infestation	No	remove
9674	bigleaf maple	*Acer macrophyllum*	14	12	F	extensive ivy infestation	No	remove
9675	bigleaf maple	*Acer macrophyllum*	8	10	P	extensive ivy infestation	No	remove
9676	bigleaf maple	*Acer macrophyllum*	12	12	F	extensive ivy infestation	No	remove
9677	bigleaf maple	*Acer macrophyllum*	10	12	F	extensive ivy infestation	No	remove
9678	bigleaf maple	*Acer macrophyllum*	10	12	F	extensive ivy infestation	No	remove
9679	bigleaf maple	*Acer macrophyllum*	3x14	14	P	extensive ivy infestation	No	remove
9680	bigleaf maple	*Acer macrophyllum*	16	14	F	extensive ivy infestation	No	remove
9681	bigleaf maple	*Acer macrophyllum*	18	14	F	extensive ivy infestation	No	remove
9682	bigleaf maple	*Acer macrophyllum*	12	12	F	extensive ivy infestation	No	remove
9683	bigleaf maple	*Acer macrophyllum*	20	16	F	extensive ivy infestation	No	remove
9684	bigleaf maple	*Acer macrophyllum*	20	16	F	extensive ivy infestation	No	remove
9685	bigleaf maple	*Acer macrophyllum*	14	12	F	extensive ivy infestation	No	remove
9686	bigleaf maple	*Acer macrophyllum*	10	12	P	extensive ivy infestation	No	remove
9687	bigleaf maple	*Acer macrophyllum*	18	16	F	extensive ivy infestation	No	remove
9692	bigleaf maple	*Acer macrophyllum*	3x14	14	P	extensive ivy infestation	No	remove
9693	bigleaf maple	*Acer macrophyllum*	10	12	P	extensive ivy infestation	No	remove
9694	bigleaf maple	*Acer macrophyllum*	20	16	F	extensive ivy infestation	No	remove
9696	bigleaf maple	*Acer macrophyllum*	24	16	F	not evaluated	Off-site	protect
9698	bigleaf maple	*Acer macrophyllum*	12	12	F	not evaluated	Off-site	protect
9701	bigleaf maple	*Acer macrophyllum*	2x10	12	P	extensive ivy infestation	No	remove
9706	bigleaf maple	*Acer macrophyllum*	5x16	16	P	not evaluated	Off-site	protect
9708	bigleaf maple	*Acer macrophyllum*	20	16	F	not evaluated	Off-site	protect
9709	bigleaf maple	*Acer macrophyllum*	16	14	F	not evaluated	Off-site	protect
9711	bigleaf maple	*Acer macrophyllum*	16	14	F	extensive ivy infestation	No	retain
9713	bigleaf maple	*Acer macrophyllum*	24	16	F	extensive ivy infestation	No	remove
9714	bigleaf maple	*Acer macrophyllum*	16	14	F	extensive ivy infestation	No	remove

No.	Common Name	Species Name	DBH*	C-Rad^	Cond#	Defects and Comments	Sig?	Treatment
9715	bigleaf maple	*Acer macrophyllum*	2x16	16	F	extensive ivy infestation	No	remove
9721	bigleaf maple	*Acer macrophyllum*	10	12	P	extensive ivy infestation	No	remove
9722	bigleaf maple	*Acer macrophyllum*	14	12	F	extensive ivy infestation	No	remove
9723	bigleaf maple	*Acer macrophyllum*	12	12	F	extensive ivy infestation	No	remove
9724	bigleaf maple	*Acer macrophyllum*	12	12	F	extensive ivy infestation	No	remove
9726	bigleaf maple	*Acer macrophyllum*	12	12	F	extensive ivy infestation	No	remove
9727	bigleaf maple	*Acer macrophyllum*	12	12	F	extensive ivy infestation	No	remove
9728	bigleaf maple	*Acer macrophyllum*	16	14	F	extensive ivy infestation	No	remove
9729	bigleaf maple	*Acer macrophyllum*	12	12	F	extensive ivy infestation	No	remove
9730	bigleaf maple	*Acer macrophyllum*	2x8	12	P	extensive ivy infestation	No	remove
9731	bigleaf maple	*Acer macrophyllum*	2x8	12	P	extensive ivy infestation	No	remove
9732	bigleaf maple	*Acer macrophyllum*	16	14	F	extensive ivy infestation	No	remove
9733	bigleaf maple	*Acer macrophyllum*	3x16	14	P	extensive ivy infestation	No	remove
9734	bigleaf maple	*Acer macrophyllum*	12	12	P	extensive ivy infestation	No	remove
9735	bigleaf maple	*Acer macrophyllum*	3x12	12	F	extensive ivy infestation	No	remove
9736	bigleaf maple	*Acer macrophyllum*	2x10	12	F	extensive ivy infestation	No	remove
9737	bigleaf maple	*Acer macrophyllum*	2x10	12	F	extensive ivy infestation	No	remove
9738	bigleaf maple	*Acer macrophyllum*	2x14	12	F	extensive ivy infestation	No	remove
9746	bigleaf maple	*Acer macrophyllum*	14	12	F	extensive ivy infestation	No	remove
9747	bigleaf maple	*Acer macrophyllum*	16	14	F	extensive ivy infestation	No	remove
9748	bigleaf maple	*Acer macrophyllum*	16	14	F	extensive ivy infestation	No	remove
9749	bigleaf maple	*Acer macrophyllum*	2x10	12	F	extensive ivy infestation	No	remove
9750	bigleaf maple	*Acer macrophyllum*	12	12	F	extensive ivy infestation	No	remove
9751	bigleaf maple	*Acer macrophyllum*	10	12	P	extensive ivy infestation	No	remove
9752	Douglas-fir	*Pseudotsuga menziesii*	30	16	F	not evaluated	Off-site	protect
9753	bigleaf maple	*Acer macrophyllum*	16	25	F	not evaluated	Off-site	protect
9754	bigleaf maple	*Acer macrophyllum*	16	14	F	not evaluated	Off-site	protect

No.	Common Name	Species Name	DBH*	C-Rad^	Cond#	Defects and Comments	Sig?	Treatment
9756	Douglas-fir	*Pseudotsuga menziesii*	36	16	G	no major defects	Yes	remove
9757	bigleaf maple	*Acer macrophyllum*	24	16	G	only suitable for retention with adjacent trees	Yes	remove
9758	bigleaf maple	*Acer macrophyllum*	24	16	G	only suitable for retention with adjacent trees	Yes	remove
9759	grand fir	*Abies grandis*	38	16	G	no major defects	Yes	remove
9769	bigleaf maple	*Acer macrophyllum*	4x16	16	F	extensive ivy infestation	No	remove
9770	bigleaf maple	*Acer macrophyllum*	8	10	P	extensive ivy infestation	No	remove
9771	bigleaf maple	*Acer macrophyllum*	12	12	F	extensive ivy infestation	No	remove
9896	bigleaf maple	*Acer macrophyllum*	6	8	F	not evaluated	No	retain
9897	bigleaf maple	*Acer macrophyllum*	12	12	F	not evaluated	No	retain
9901	bigleaf maple	*Acer macrophyllum*	16	14	G	not evaluated	No	retain
10072	bigleaf maple	*Acer macrophyllum*	12	12	F	not evaluated	No	retain
10098	bigleaf maple	*Acer macrophyllum*	28	16	G	no major defects	Yes	remove
10101	bigleaf maple	*Acer macrophyllum*	24	16	G	no major defects	Yes	remove
10113	grand fir	*Abies grandis*	60	18	G	no major defects	Yes	remove
20000	Douglas-fir	*Pseudotsuga menziesii*	14	8	P	suppressed, not viable	No	remove
20001	Douglas-fir	*Pseudotsuga menziesii*	46	28	G	old broken top, moderate crown structure, ivy at base and up lower trunk	Yes	remove
20002	Douglas-fir	*Pseudotsuga menziesii*	34	22	F	codominant crown class, old broken top, moderate crown structure	Yes	remove

***DBH** is tree diameter measured at breast height, 4.5-feet above the ground level (inches); codominant trunks splitting below DBH are measured individually and separated by a comma, except for codominant stems of equal size are noted as quantity x size. **^C-Rad** is the average crown radius measured in feet. **#Cond** is an arborist assigned rating to generally describe the condition of individual trees as follows- Dead; Poor; Fair; or Good Condition. **Sig?** asks whether or not the tree is considered potentially significant, either Yes (likely significant) or No (not considered significant).

AEGIS PLUS®

COMMERCIAL STRENGTH ORNAMENTAL STEEL

- .75"sq. x 17ga. PICKETS
- 1.5"w. x 1.5"h. x 14ga. FORERUNNER® RAILS
- 2.5"sq. x 12ga. / 3"sq. x 12ga. POSTS
- 3', 3½', 4', 5', 6', 7' & 8' FENCE HEIGHTS
- 2 & 3 RAIL OPTIONS

CLASSIC™

TIMELESS & ELEGANT

Extended pickets that culminate to an *arrow-pointed spear* capture the look of old style wrought iron fencing. *Single or double swing & slide gates* matching this fence style are also available.

MAJESTIC™

SLEEK & MODERN

The *flush top rail* projects a more *modern, streamlined look* that beautifully accents flowers and shrubs when used as border landscaping. *Single or double swing & slide gates* matching this fence style are also available.

GENESIS™

PERSONALIZED & SECURE

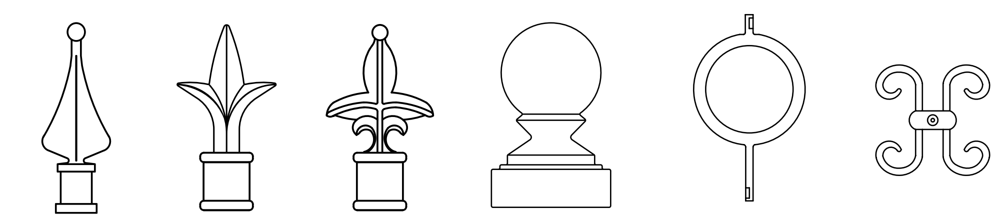
Extended flat-topped pickets serve as a base for your *choice of accent finials* providing a customized design. *Single or double swing & slide gates* matching this fence style are also available.

WARRIOR™

DISTINCTIVE & CLASSIC

Alternating heights of extended pickets that culminate to an *arrow-pointed spear* bring a hint of old-world elegance to this fence style. *Single or double swing & slide gates* matching this fence style are also available.

ADORNMENTS



QUAD FLARE | TRIAD | ROYALTY | BALL POST CAP | RING *(internally secured)* | BUTTERFLY SCROLL

COLOR OPTIONS

BLACK | BRONZE | DESERT SAND | WHITE

Reservoir exterior wall shotcrete finish - Example photo

PRELIMINARY STORMWATER MANAGEMENT REPORT

FOR

BOLTON RESERVOIR REPLACEMENT PROJECT
CITY OF WEST LINN, OREGON

MAY 2015

OWNER:
City of West Linn,
Public Work Department
22500 Salamo Road
West Linn, OR 97068

Lance Calvert
Public Works Director
503-722-5516

ENGINEER:
Michael L. McKillip, P. E.
Murray, Smith & Associates, Inc.
121 SW Salmon, Suite 900
Portland, OR 97204
Ph: (503) 225-9010

PROJECT LOCATION:
6111 Skyline Drive
West Linn, OR 97

PERMIT NUMBERS:
Conditional Use and Design Review PA-15-05

SUBMITTAL DATE:
May 2015

SECTION 2
DESIGNER'S CERTIFICATION AND STATEMENT

I hereby certify that this Stormwater Management Report for the Bolton Reservoir Replacement project has been prepared by me and meets minimum standards of the City of West Linn and normal standards of engineering practice. I hereby acknowledge and agree that the jurisdiction does not and will not assume liability for the sufficiency, suitability, or performance of drainage facilities designed by me.

SECTION 3
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1. COVER SHEET

2. DESIGNER'S CERTIFICATION AND STATEMENT

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5. METHODOLOGY

6. ANALYSIS
- Stormwater Narrative and Analysis
- HydroCAD Runoff and Detention Calculations
- Web Soil Survey Information
- Supporting Calculations <to follow, as needed>

7. ENGINEERING CONCLUSIONS

8. STORMWATER FACILITY DETAILS AND EXHIBITS
- Drainage Basin Map from 2006 Surface Water Management Plan
- Sheet CA-2 Demolition and Tree Removal Plan <to follow>
- Sheet CA-3 Grading Plan <to follow>
- Sheet CA-6 Site Piping Plan <to follow>
- Sheet CA- 16 Stormwater Detention Facility and Details <to follow>
- Sheet CA-17 Stormwater Swale Details <to follow>
- Sheet LA-1 Landscape Plan <to follow>
- Sheet LA-2 Planting Details <to follow>

9. OPERATIONS AND MAINTENANCE

<to follow>

10. ADDITIONAL FORMS (N/A)

11. ASSOCIATED REPORTS SUBMITTED
- Geotechnical Engineering Report, prepared by GRI <to follow, see land use application attachments>
- Arborist Report, prepared by Morgan Holen & Associates, LLC <to follow, see land use application attachments>

SECTION 4
PROJECT OVERVIEW AND DESCRIPTION

The existing property, located at 6111 Skyline Drive in the City of West Linn, is the site of the City's main drinking water supply reservoir. The Bolton Reservoir was constructed as an open-air water supply reservoir circa 1915. A pump station building was added (date unknown) and subsequently taken out of service and a new pump station was added in the 1980s. A synthetic reservoir cover was added in 1996. The reservoir and cover are both exhibiting problems due to the age of the facilities. The reservoir will be replaced with a larger volume pre-stressed concrete structure meeting current design standards and accommodating the water system's storage needs.

The project includes demolition of the old pump station structures and the 2.5 million gallon reservoir, installation of a partially buried 4.0 million gallon reservoir and associated piping and valving. Other site improvements include site grading, stormwater management facilities, access roads, landscaping and replacement of the exiting Bolton Pump Station roof.

Topography within the project site varies from gently northward sloping areas to the south to a slope as steep as 1H:1V along the north property boundary. The proposed site plan includes regrading the steep slope to 3H:1V to match the existing hillside slope downhill of the project site.

The 2006 Surface Water Management Plan (SWMP) delineated the two basins to which the current site contributes runoff. The existing drainage is primarily vegetated area overland runoff which ultimately reaches the Bolton Creek drainage system. The Bolton Creek drainage basin is a 113 acres basin discharging to the Willamette River. The SWMP further identified needed pipe capacity improvement downstream of the Highway 43 crossing. The existing reservoir cover collects stormwater which is pumped to Bolton Creek. The proposed redevelopment will not increase the peak runoff contributing to Bolton Creek.

The paved area around the existing pump station and the roof runoff from the three buildings is collected and conveyed into the municipal stormwater system along Skyline Circle. This runoff is conveyed to the Maddax Creek drainage basin, which drains approximately 138 acres. The SWMP identified two capacity deficiencies downstream of the project site. The impervious and total area contributing to the Maddax Creek drainage basin will be reduced as part of this project to ensure that the peak runoff is not increased.

SECTION 5
METHODOLOGY

The current site has three functionally distinct drainage areas. The paved area around the pump stations is collected in storm sewer and conveyed to the east down the municipal storm sewer system in Skyline Circle. The proposed site redevelopment will retain this function and not increase the impervious area or drainage area contributing to these stormwater facilities. This area includes approximately 7,300 square feet of paved impervious area, building roof runoff, and other runoff area.

The second functional area is the existing reservoir cover, which intercepts rainwater from entering the reservoir and is collected and pumped to the stormwater manhole at the north edge of the property. This manhole conveys flow via pipe to Bolton Creek, approximately 250 feet to the north. The reservoir cover sump pumps discharge at approximately 70 gpm (0.16 cfs).

The third functional area, which includes the majority of the site, is comprised of a mix of wooded areas, gravel access road, and grassy areas which drains overland to the slope at the north portion of the site. This runoff appears to become naturally channelized along the slope and ultimately contributes to a drainage ditch which routes water to Bolton Creek.

The proposed development is required to not increase the peak runoff rate to both the streams receiving piped runoff and the parcels receiving overland flow. Specifically, the post-development 2-, 5-, 10- and 25-year storm runoff will not exceed the pre-development storm of the same recurrence interval. Additionally, when discharging via pipe to streams, the post-development 2-year storm will not exceed one-half the pre-development 2-year storm.

As identified in the Geotechnical Engineering Report prepared by GRI, included in Section 11 of this report, control of groundwater is important for both the reservoir foundation conditions as well as improving the slope stability along the steep slope to the north. Consequently, stormwater management approaches that use infiltration are not appropriate for this site. As such, the impervious areas and any channelized runoff will be directed to a proposed stormwater pond with an outlet control structure to provide for stormwater detention. The stormwater pond will be lined to prevent excessive infiltration. The outlet control structure will limit discharge from the pond to the existing reservoir sump pump discharge rate.

Water quality treatment is required for the reservoir concrete roof impervious area and the new impervious paved access road. The roof runoff will be collected by a French drain and conveyed to the stormwater pond. Discharge from the pond will be routed to a proposed water quality swale. The swale will discharge to a proposed manhole and be conveyed to Bolton Creek in a new storm sewer pipe. The existing drainage piping is old and will be abandoned.

SECTION 6
ANALYSIS

Design Storm

The rainfall data used to develop hydrographs and calculate runoff and allowable release rates for the design storms were taken from the "Portland Stormwater Management Manual" which uses the NRCS Type 1A 24-hour storm distribution. The rainfall depths are shown in Table 6-1.

Table 6-1
Design Storm Rainfall Depths

Recurrence Interval, Years	24-Hour Rainfall Depths, Inches
2	2.4
5	2.9
10	3.4
25	3.9
100	4.4

Pre-development Site Characterization

HydroCAD was used to develop the hydrographs and to size the underground detention facilities and outlet control structures along with the design storms in Table 6-1. The site soils are SCS Classification 13B, Cascade Silt Loam, 3 to 8 percent slopes, which is a classification of USDA hydrologic soil group C. The county soil data is included in this section.

The pre-development site consists of approximately 0.65 acres of steeply sloped vegetated land at an average slope of 50 percent, 0.78 acres of wooded area at 5 percent slope, and 1.80 acres of grassy land at approximately 5 percent slope. Curve Number (CN) of 73 was assigned for the wooded area, and a CN of 74 was assigned to the grassy land. All soils used hydrologic soil group C.

Post-development Site Characterization

The post-development site includes a mix of gravel areas, paved access, and impervious reservoir concrete roof which are collected and routed to the stormwater detention pond. Areas contributing to the stormwater pond detention include all impervious areas as well as pervious areas south of the reservoir structure. Impervious areas are assigned a CN of 98, gravel areas a CN of 89, grassy areas a CN of 74 and 73 for wooded areas.

Pervious areas north of the proposed reservoir, existing pump station, and proposed pond, which will continue to drain overland, are assigned a CN of 74 for grassy areas and 89 for gravel areas.

The post-development area contributing to each drainage location is summarized by ground characteristics in Table 6-2.

Table 6-2
Drainage Area Summary

Area Description	Area (acres)	CN
Drainage to Skyline Circle		
Pump Station Roof	0.034	98
Paved Area	0.066	98
Subtotal	**0.101**	
Overland Flow		
Gravel Area	0.041	89
Grassy Area	0.897	74
Subtotal	**0.938**	
Stormwater Facilities to Bolton Creek		
Reservoir Roof	0.510	98
Paved Access	0.265	98
Gravel Access	0.384	89
Wooded Area	0.358	73
Grassy Area	0.674	74
Subtotal	**2.191**	
Total Area	3.230	

Runoff Rates

The target runoff rate is that of the pre-developed site condition for the 2-, 5-, 10- and 25-year storm events. The runoff at the current level of development exceeds that target. In addition, the current site runoff consists of piped runoff to Bolton Creek, and overland runoff to the downhill lots. The site constraints limit the location and size of the proposed stormwater detention pond which will ensure that the piped runoff to Bolton Creek does not exceed the current peak runoff rate. As such, the proposed redevelopment will return the peak runoff to below the pre-development runoff condition for all design storms except the 2-year storm, which is marginally exceeded. However, the proposed redevelopment peak runoff will reduce the 2-year peak runoff rate to 60 percent of the existing condition. The existing, pre-development and post-development peak runoff rates for the total site are presented in Table 6-3. These rates do not include the existing paved area around the existing pump station which drains to the east and will not be redeveloped as part of this project.

As shown in Table 6-4, under the 2-year storm, the developed overland flow component is much smaller than the pre-development flows; and the post-development flows to Bolton Creek are much smaller than the existing condition.

Table 6-5 summarizes the outlet control at the stormwater detention pond. The 1-inch diameter orifice is the minimum recommended size and the configuration of the pond and outlet structure is established to control the larger design storm events. The peak runoff from the 100-year storm will be mitigated and the detention, water quality, and piping facilities are sized and equipped with overflow features to convey the peak runoff.

Table 6-3
Peak Runoff Rates

Recurrence Interval, Years	Peak Runoff Rate, cfs		
	Existing Development[1]	Pre-Development	Post-Development[2]
2	0.28	0.15*3	0.16*3
5	0.41	0.30	0.24
10	0.56	0.48	0.36
25	0.73	0.69	0.48
100	0.91	0.91	0.76

Notes: 1) Includes existing reservoir cover runoff.
2) Includes overland runoff and discharge to the municipal stormwater system.
*3) See text for discussion of runoff control limitations.

Table 6-4
Overland Peak Runoff Rates and Peak Creek Discharge

Recurrence Interval, Years	Overland Runoff		Creek Discharge	
	Pre-Development	Post-Development	Existing Condition	Post-Development
2	0.15	0.08	0.16	0.08
5	0.30	0.15	0.16	0.09
10	0.48	0.23	0.16	0.13
25	0.69	0.32	0.16	0.16
100	0.91	0.41	0.16	0.35

Table 6-5
Outlet Control Structure Summary

Feature	Elevation, feet
Top of freeboard	439.50
Emergency overflow elevation	438.00
1.5-inch orifice	437.00
1.0-inch orifice	435.00
1.0-inch orifice	430.50
Detention Pond Bottom	430.00
Bottom of outlet sump	427.00

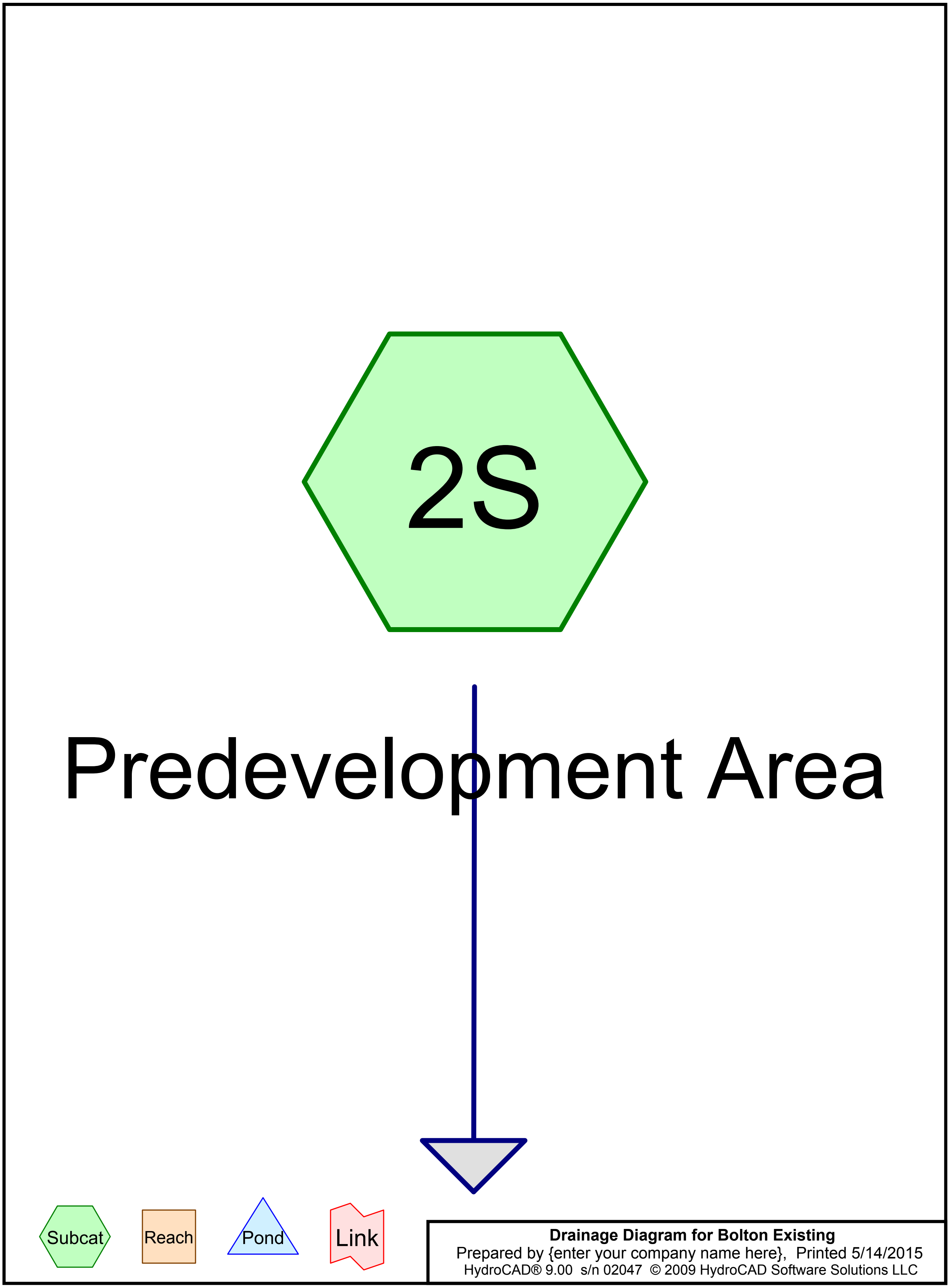
Water Quality Facilities

The water quality swale is designed in accordance with the Portland Stormwater Management Manual Grassy Swale facility performance approach. The maximum design velocity is limited to 9 feet per second with a Manning 'n' value of 0.25. Minimum hydraulic residence time of 9 minutes for the 2-yr design storm event.

The swale can also pass the flow of a 25-year event and will be equipped with an emergency overflow facility in the event that the outlet becomes clogged.

Table 6-6
Swale Characteristics

Feature	Value
Channel longitudinal slope	1 Percent
Bottom width	3 feet
Channel side slope	3H:1V
Manning's 'n' value	0.25
Design flow rate	0.08 cubic feet per second
Max water depth	2 inches
Total depth with freeboard	18 inches
Velocity	0.15 feet per second
Swale length	110 feet
Travel time	12.1 minutes



Drainage Diagram for Bolton Existing
Prepared by {enter your company name here}, Printed 5/14/2015
HydroCAD® 9.00 s/n 02047 © 2009 HydroCAD Software Solutions LLC

Area Listing (selected nodes)

Area (acres)	CN	Description (subcatchment-numbers)
0.780	73	Wooded, fair, HSG C (2S)
2.450	74	Grass, fair, HSG C (2S)
3.230		**TOTAL AREA**

Summary for Subcatchment 2S: Predevelopment Area

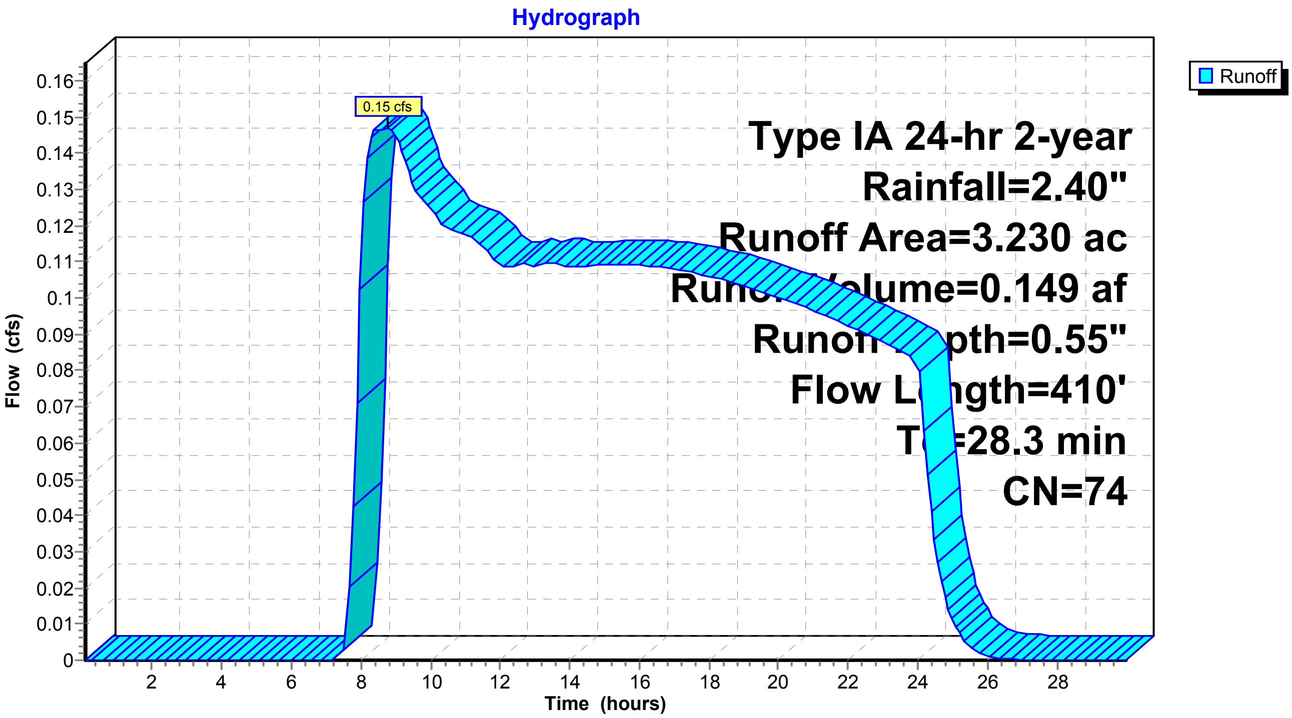
Runoff = 0.15 cfs @ 8.77 hrs, Volume= 0.149 af, Depth= 0.55"

Runoff by SBUH method, Time Span= 0.08-29.98 hrs, dt= 0.10 hrs
Type IA 24-hr 2-year Rainfall=2.40"

	Area (ac)	CN	Description
*	2.450	74	Grass, fair, HSG C
*	0.780	73	Wooded, fair, HSG C
	3.230	74	Weighted Average
	3.230		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
27.5	300	0.0500	0.18		**Sheet Flow, Offsite Area** Grass: Dense n= 0.240 P2= 2.40"
0.3	60	0.5000	3.54		**Shallow Concentrated Flow, Yard** Woodland Kv= 5.0 fps
0.5	50	0.0500	1.57		**Shallow Concentrated Flow, Flat, shallow conc grass** Short Grass Pasture Kv= 7.0 fps
28.3	410	Total			

Subcatchment 2S: Predevelopment Area



Summary for Subcatchment 2S: Predevelopment Area

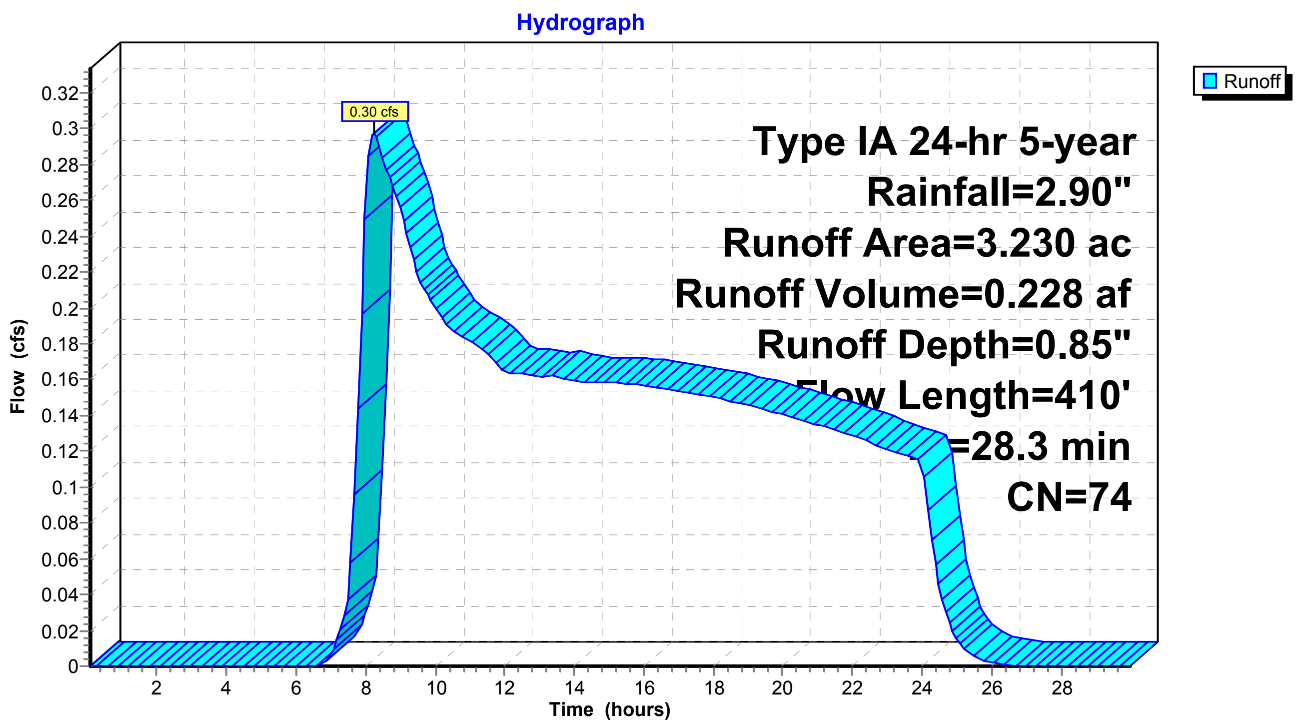
Runoff = 0.30 cfs @ 8.24 hrs, Volume= 0.228 af, Depth= 0.85"

Runoff by SBUH method, Time Span= 0.08-29.98 hrs, dt= 0.10 hrs
Type IA 24-hr 5-year Rainfall=2.90"

	Area (ac)	CN	Description
*	2.450	74	Grass, fair, HSG C
*	0.780	73	Wooded, fair, HSG C
	3.230	74	Weighted Average
	3.230		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
27.5	300	0.0500	0.18		**Sheet Flow, Offsite Area** Grass: Dense n= 0.240 P2= 2.40"
0.3	60	0.5000	3.54		**Shallow Concentrated Flow, Yard** Woodland Kv= 5.0 fps
0.5	50	0.0500	1.57		**Shallow Concentrated Flow, Flat, shallow conc grass** Short Grass Pasture Kv= 7.0 fps
28.3	410	Total			

Subcatchment 2S: Predevelopment Area



Summary for Subcatchment 2S: Predevelopment Area

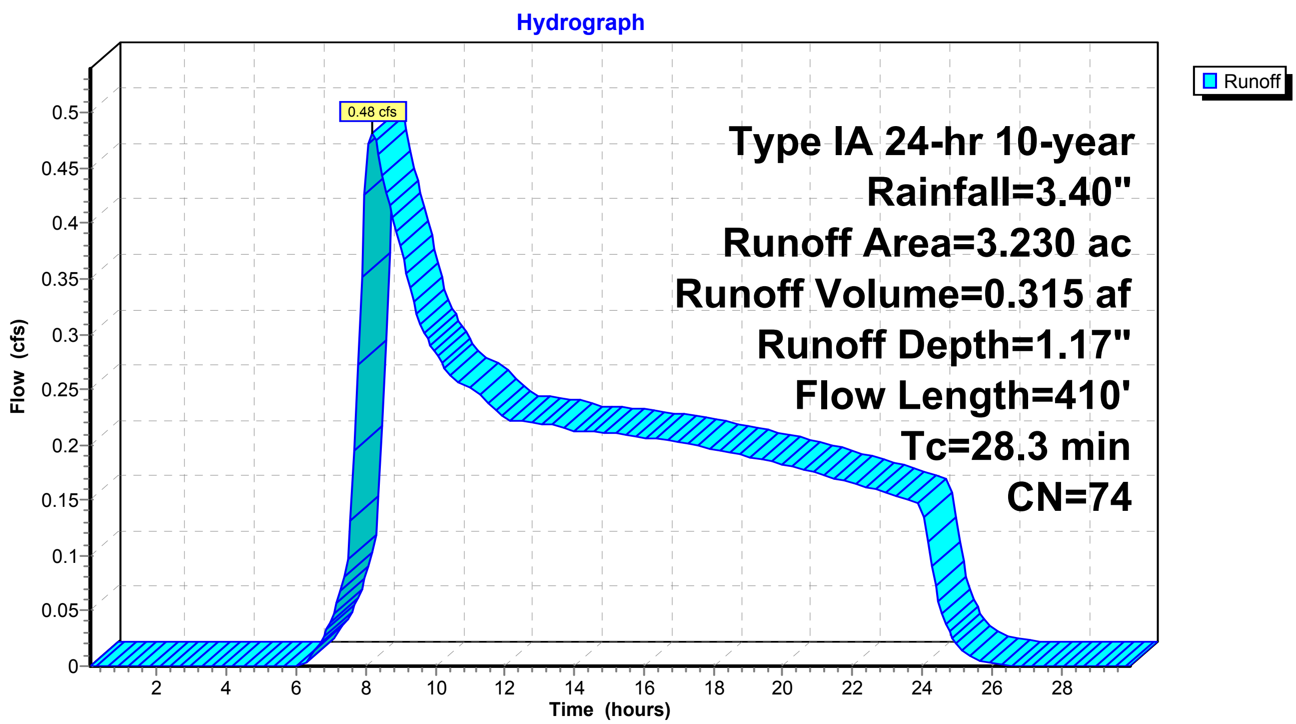
Runoff = 0.48 cfs @ 8.19 hrs, Volume= 0.315 af, Depth= 1.17"

Runoff by SBUH method, Time Span= 0.08-29.98 hrs, dt= 0.10 hrs
Type IA 24-hr 10-year Rainfall=3.40"

	Area (ac)	CN	Description
*	2.450	74	Grass, fair, HSG C
*	0.780	73	Wooded, fair, HSG C
	3.230	74	Weighted Average
	3.230		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
27.5	300	0.0500	0.18		**Sheet Flow, Offsite Area** Grass: Dense n= 0.240 P2= 2.40"
0.3	60	0.5000	3.54		**Shallow Concentrated Flow, Yard** Woodland Kv= 5.0 fps
0.5	50	0.0500	1.57		**Shallow Concentrated Flow, Flat, shallow conc grass** Short Grass Pasture Kv= 7.0 fps
28.3	410	Total			

Subcatchment 2S: Predevelopment Area



Summary for Subcatchment 2S: Predevelopment Area

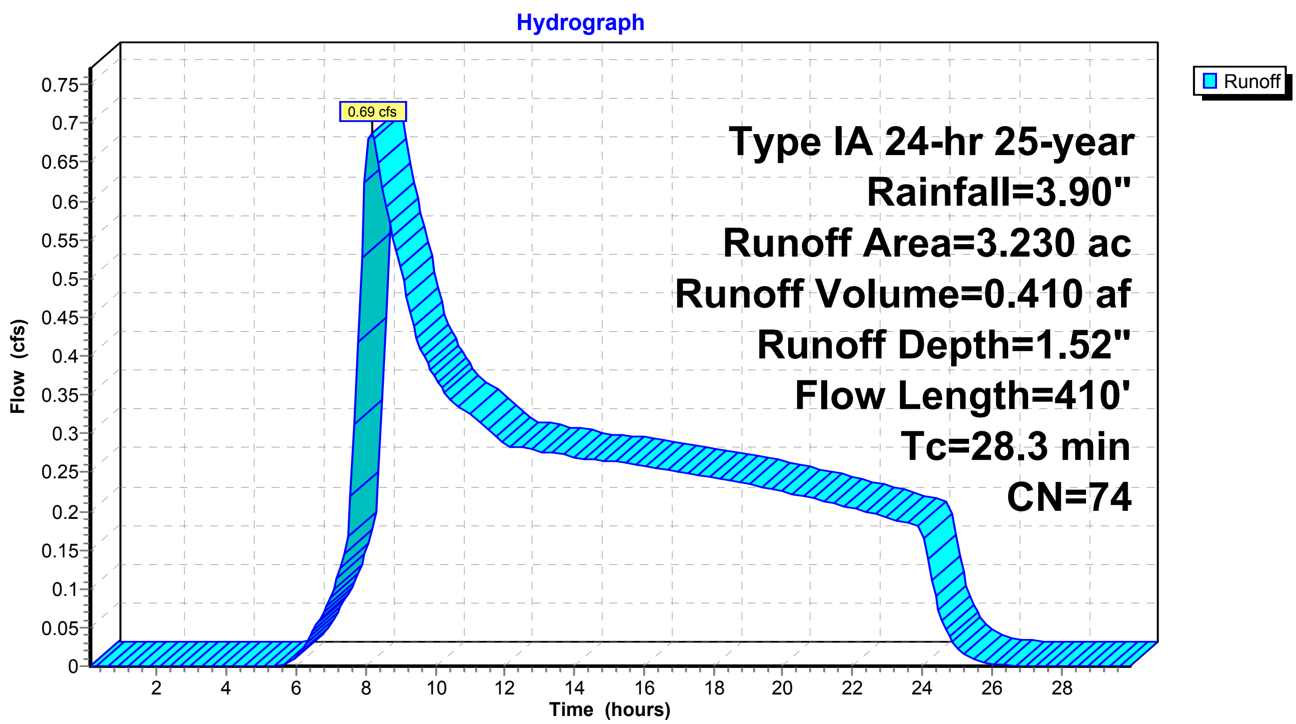
Runoff = 0.69 cfs @ 8.16 hrs, Volume= 0.410 af, Depth= 1.52"

Runoff by SBUH method, Time Span= 0.08-29.98 hrs, dt= 0.10 hrs
Type IA 24-hr 25-year Rainfall=3.90"

	Area (ac)	CN	Description
*	2.450	74	Grass, fair, HSG C
*	0.780	73	Wooded, fair, HSG C
	3.230	74	Weighted Average
	3.230		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
27.5	300	0.0500	0.18		**Sheet Flow, Offsite Area** Grass: Dense n= 0.240 P2= 2.40"
0.3	60	0.5000	3.54		**Shallow Concentrated Flow, Yard** Woodland Kv= 5.0 fps
0.5	50	0.0500	1.57		**Shallow Concentrated Flow, Flat, shallow conc grass** Short Grass Pasture Kv= 7.0 fps
28.3	410	Total			

Subcatchment 2S: Predevelopment Area



Summary for Subcatchment 2S: Predevelopment Area

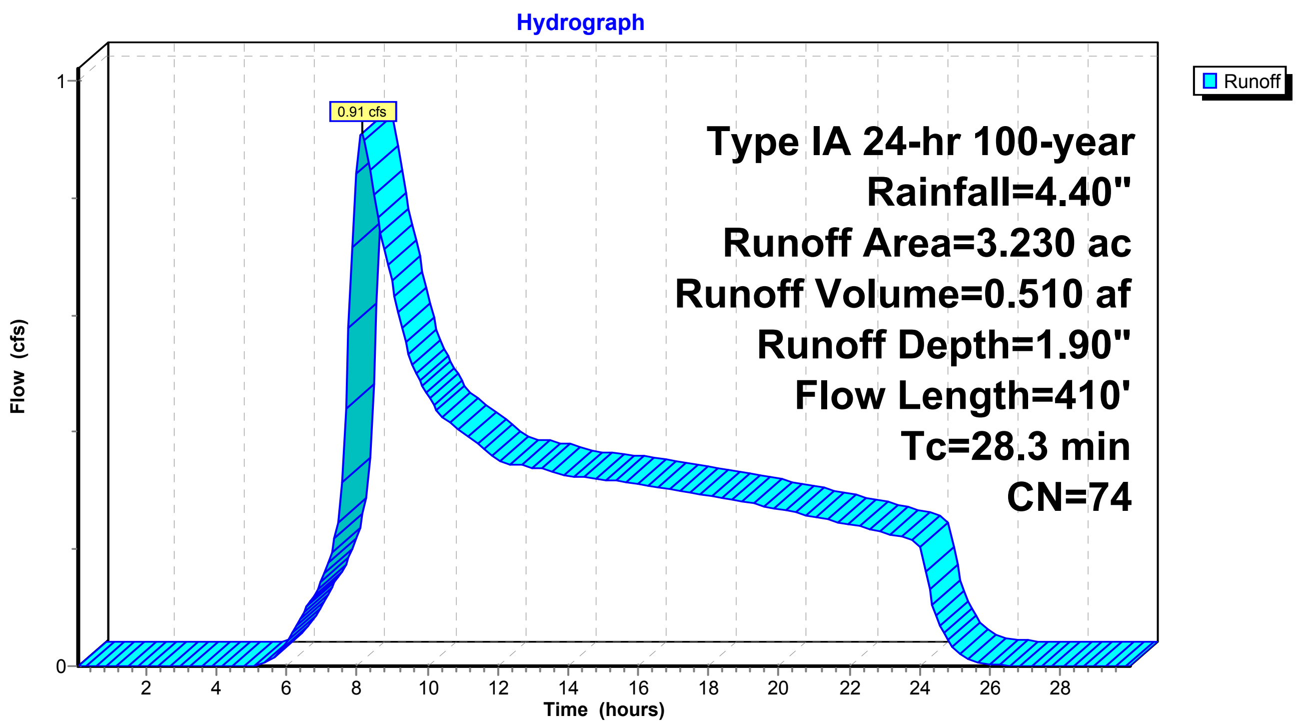
Runoff = 0.91 cfs @ 8.14 hrs, Volume= 0.510 af, Depth= 1.90"

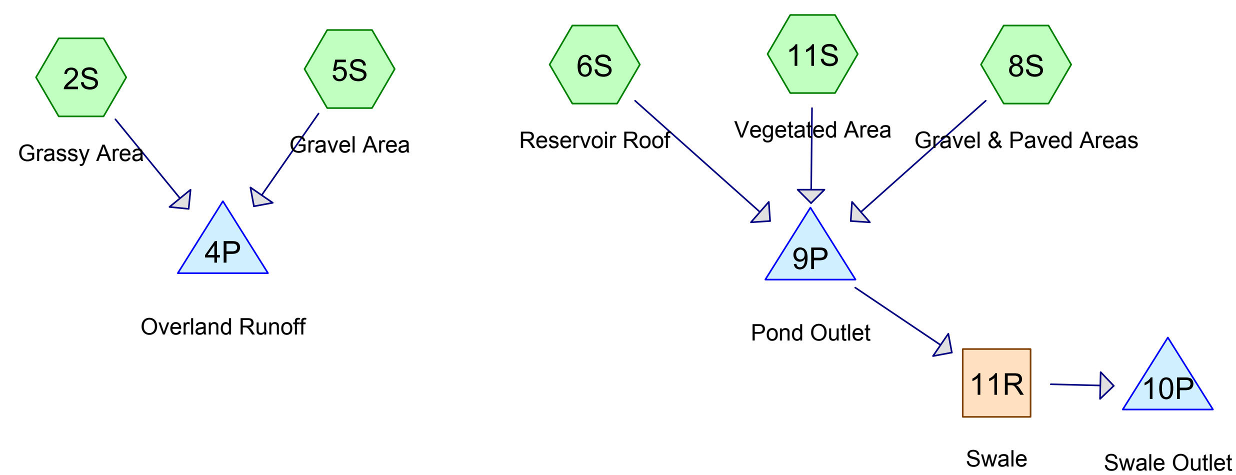
Runoff by SBUH method, Time Span= 0.08-29.98 hrs, dt= 0.10 hrs
Type IA 24-hr 100-year Rainfall=4.40"

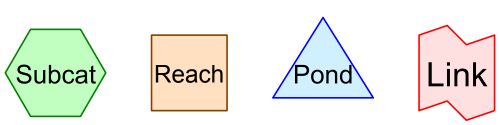
	Area (ac)	CN	Description
*	2.450	74	Grass, fair, HSG C
*	0.780	73	Wooded, fair, HSG C
	3.230	74	Weighted Average
	3.230		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
27.5	300	0.0500	0.18		**Sheet Flow, Offsite Area** Grass: Dense n= 0.240 P2= 2.40"
0.3	60	0.5000	3.54		**Shallow Concentrated Flow, Yard** Woodland Kv= 5.0 fps
0.5	50	0.0500	1.57		**Shallow Concentrated Flow, Flat, shallow conc grass** Short Grass Pasture Kv= 7.0 fps
28.3	410	Total			

Subcatchment 2S: Predevelopment Area







Drainage Diagram for Bolton Post-Prelim SWM
Prepared by {enter your company name here}, Printed 5/14/2015
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Area Listing (all nodes)

Area (acres)	CN	Description (subcatchment-numbers)
0.358	73	Woods, HSG C (11S)
1.571	74	Grass, fair, HSG C (2S, 11S)
0.384	89	Gravel access (8S)
0.041	89	Gravel access to swale, HSG C (5S)
0.265	98	Paved access (8S)
0.510	98	concrete roof (6S)
3.129		**TOTAL AREA**

Soil Listing (all nodes)

Area (acres)	Soil Group	Subcatchment Numbers
0.000	HSG A	
0.000	HSG B	
1.970	HSG C	2S, 5S, 11S
0.000	HSG D	
1.159	Other	6S, 8S
3.129		**TOTAL AREA**

Time span=0.08-48.00 hrs, dt=0.01 hrs, 4793 points
Runoff by SBUH method
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment2S: Grassy Area Runoff Area=0.897 ac 0.00% Impervious Runoff Depth=0.55"
Flow Length=110' Tc=7.0 min CN=74 Runoff=0.06 cfs 0.041 af

Subcatchment5S: Gravel Area Runoff Area=0.041 ac 0.00% Impervious Runoff Depth=1.37"
Flow Length=65' Tc=4.1 min CN=89 Runoff=0.01 cfs 0.005 af

Subcatchment6S: Reservoir Roof Runoff Area=0.510 ac 100.00% Impervious Runoff Depth=2.17"
Flow Length=360' Tc=2.2 min CN=98 Runoff=0.29 cfs 0.092 af

Subcatchment8S: Gravel & Paved Areas Runoff Area=0.649 ac 40.83% Impervious Runoff Depth=1.69"
Flow Length=290' Tc=2.0 min CN=93 Runoff=0.28 cfs 0.091 af

Subcatchment11S: Vegetated Area Runoff Area=1.032 ac 0.00% Impervious Runoff Depth=0.55"
Flow Length=350' Slope=0.0500 '/' Tc=28.0 min CN=74 Runoff=0.05 cfs 0.048 af

Reach 11R: Swale Avg. Depth=0.15' Max Vel=0.15 fps Inflow=0.08 cfs 0.190 af
n=0.250 L=110.0' S=0.0100 '/' Capacity=2.66 cfs Outflow=0.08 cfs 0.189 af

Pond 4P: Overland Runoff Inflow=0.08 cfs 0.046 af
Primary=0.08 cfs 0.046 af

Pond 9P: Pond Outlet Peak Elev=435.51' Storage=0.140 af Inflow=0.60 cfs 0.231 af
Outflow=0.08 cfs 0.190 af

Pond 10P: Swale Outlet Inflow=0.08 cfs 0.189 af
Primary=0.08 cfs 0.189 af

Total Runoff Area = 3.129 ac Runoff Volume = 0.277 af Average Runoff Depth = 1.06"
75.23% Pervious = 2.354 ac 24.77% Impervious = 0.775 ac

Summary for Subcatchment 2S: Grassy Area

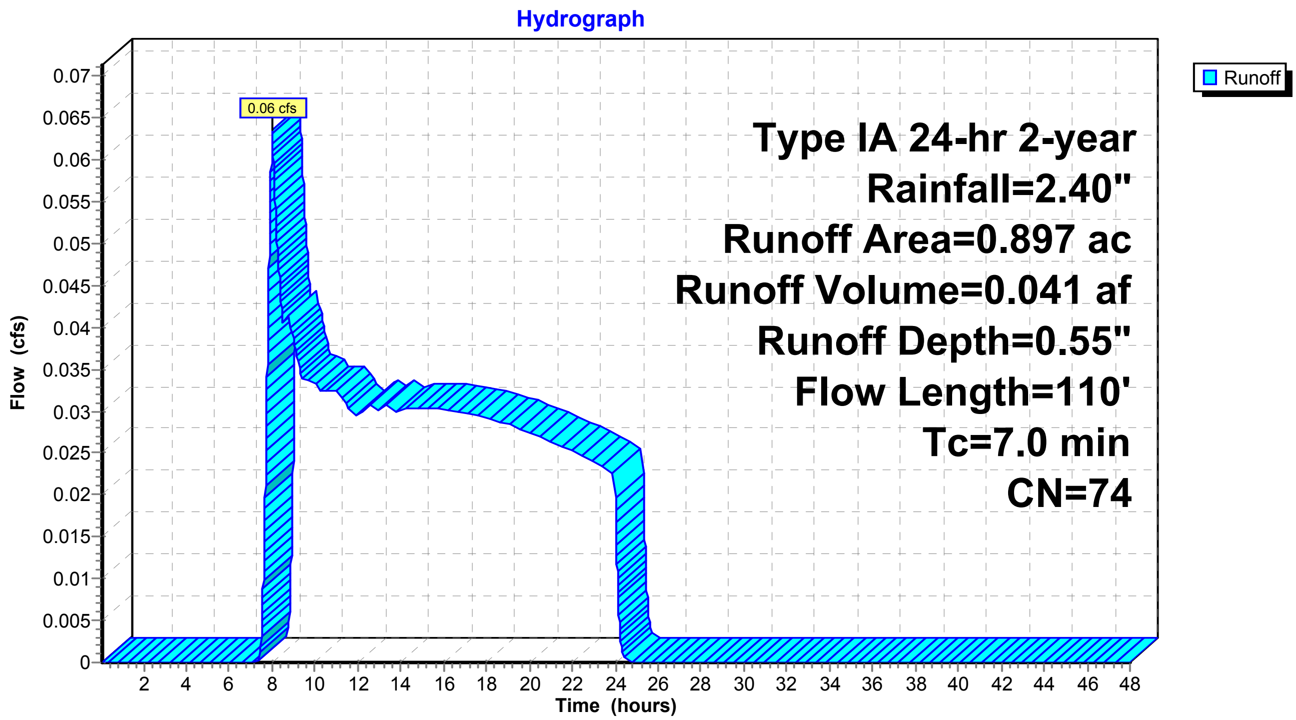
Runoff = 0.06 cfs @ 8.00 hrs, Volume= 0.041 af, Depth= 0.55"

Runoff by SBUH method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs
Type IA 24-hr 2-year Rainfall=2.40"

	Area (ac)	CN	Description
*	0.897	74	Grass, fair, HSG C
	0.897		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
3.6	60	0.3300	0.28		**Sheet Flow, steeper slope** Grass: Dense n= 0.240 P2= 2.40"
3.4	50	0.2500	0.24		**Sheet Flow, flatter slope** Grass: Dense n= 0.240 P2= 2.40"
7.0	110	Total			

Subcatchment 2S: Grassy Area



Summary for Subcatchment 5S: Gravel Area

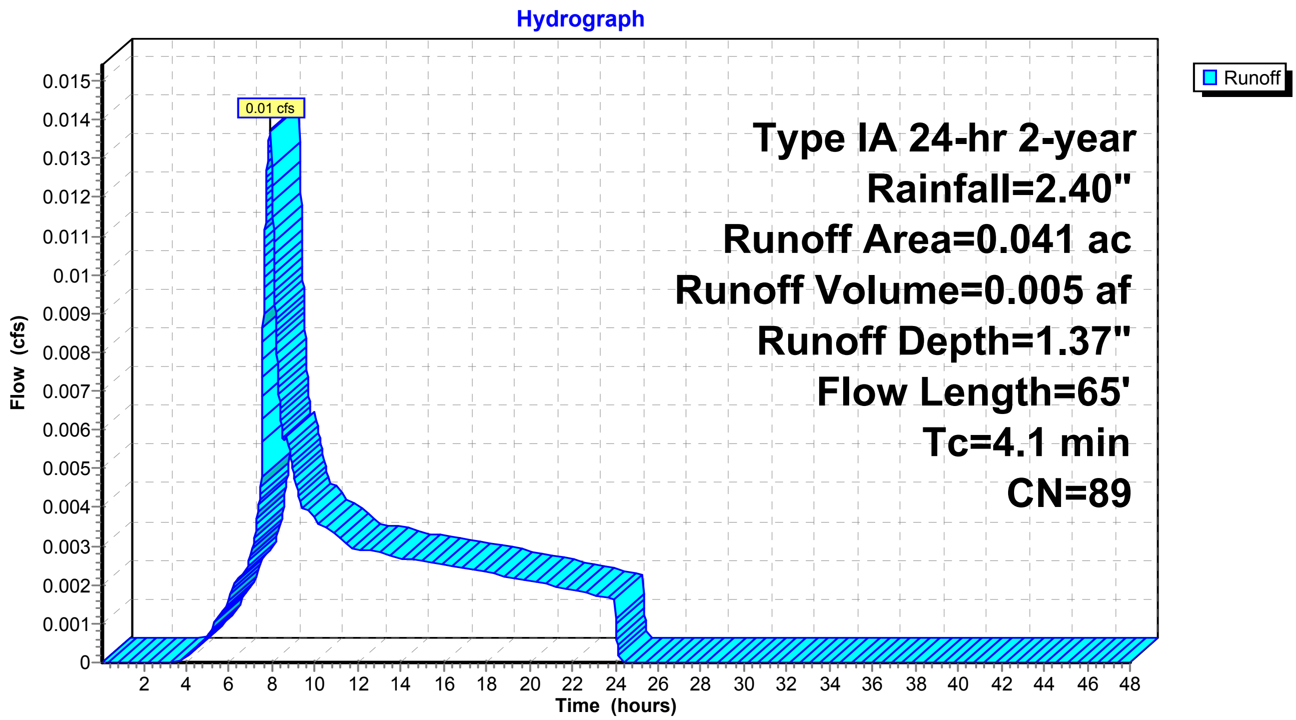
Runoff = 0.01 cfs @ 7.94 hrs, Volume= 0.005 af, Depth= 1.37"

Runoff by SBUH method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs
Type IA 24-hr 2-year Rainfall=2.40"

	Area (ac)	CN	Description
*	0.041	89	Gravel access to swale, HSG C
	0.041		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
1.0	15	0.0200	0.24		**Sheet Flow, Gravel access to pond** Fallow n= 0.050 P2= 2.40"
3.1	50	0.3300	0.27		**Sheet Flow, grass overland** Grass: Dense n= 0.240 P2= 2.40"
4.1	65	Total			

Subcatchment 5S: Gravel Area



Summary for Subcatchment 6S: Reservoir Roof

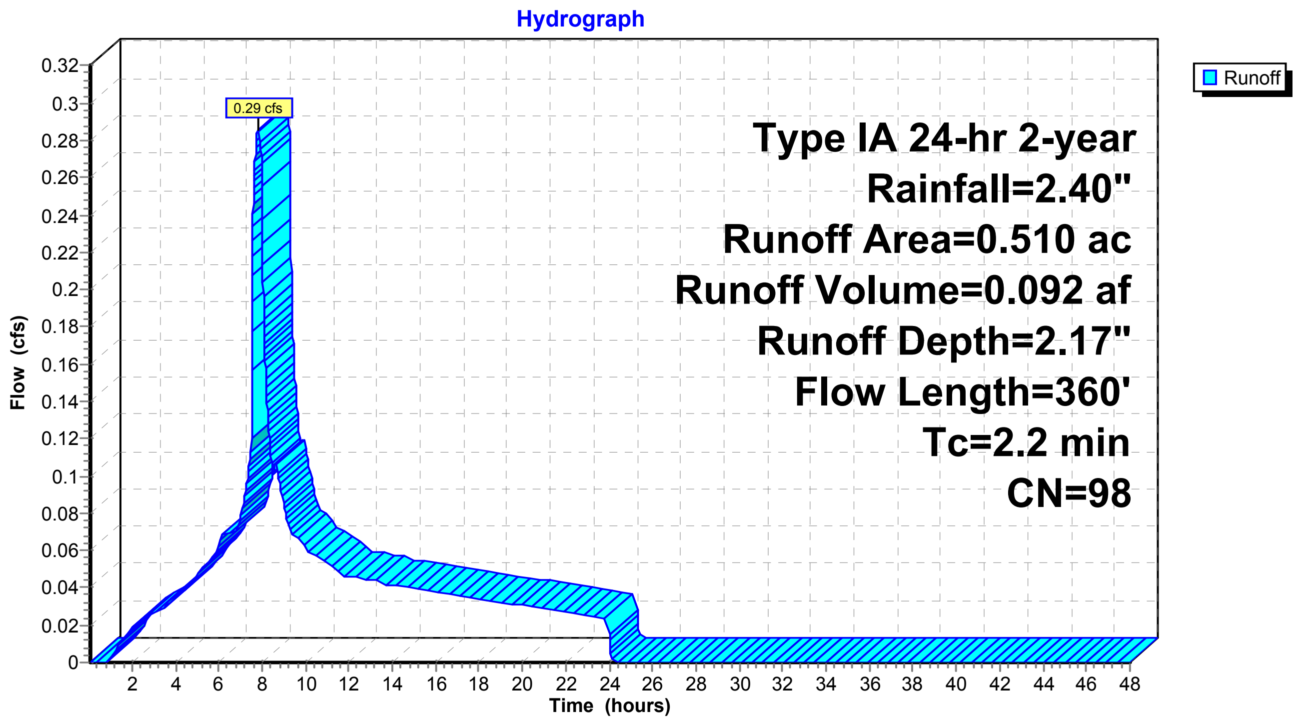
Runoff = 0.29 cfs @ 7.83 hrs, Volume= 0.092 af, Depth= 2.17"

Runoff by SBUH method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs
Type IA 24-hr 2-year Rainfall=2.40"

	Area (ac)	CN	Description
*	0.510	98	concrete roof
	0.510		100.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
1.2	85	0.0200	1.15		**Sheet Flow, Reservoir concrete roof** Smooth surfaces n= 0.011 P2= 2.40"
1.0	275	0.0750	4.41		**Shallow Concentrated Flow, gravel collection** Unpaved Kv= 16.1 fps
2.2	360	Total			

Subcatchment 6S: Reservoir Roof



Summary for Subcatchment 8S: Gravel & Paved Areas

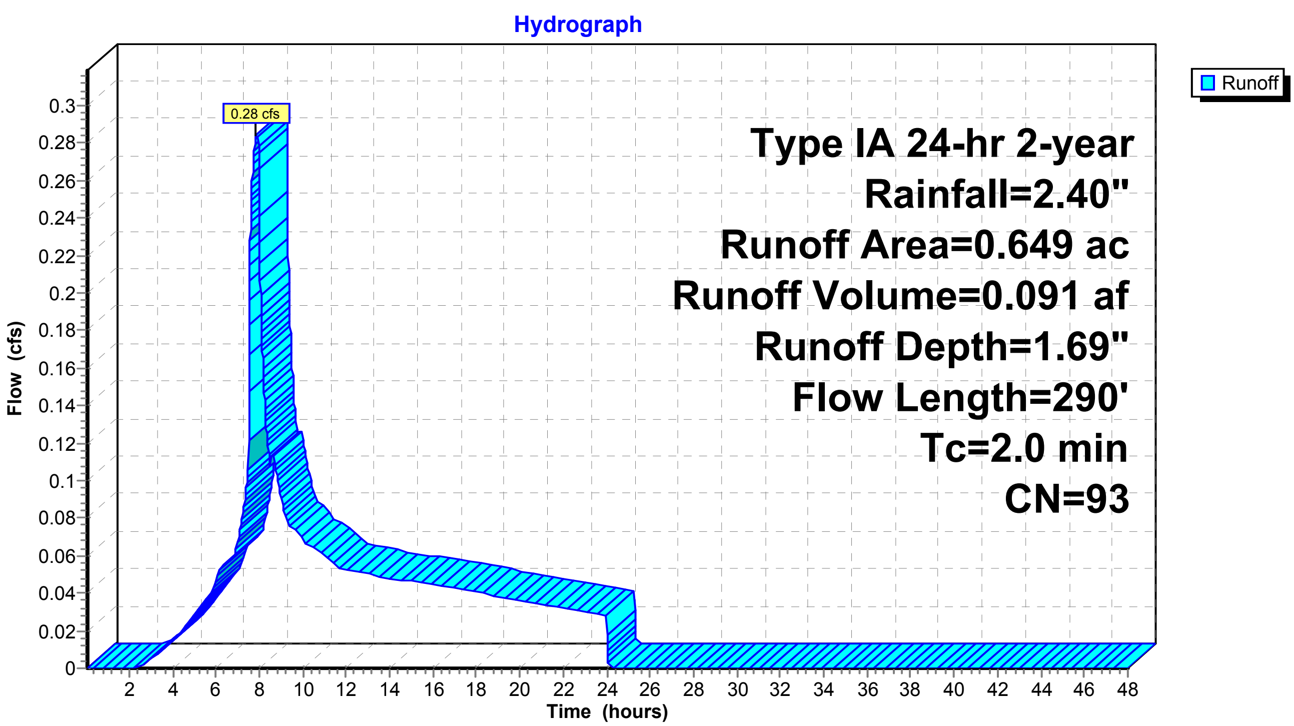
Runoff = 0.28 cfs @ 7.87 hrs, Volume= 0.091 af, Depth= 1.69"

Runoff by SBUH method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs
Type IA 24-hr 2-year Rainfall=2.40"

	Area (ac)	CN	Description
*	0.384	89	Gravel access
*	0.265	98	Paved access
	0.649	93	Weighted Average
	0.384		59.17% Pervious Area
	0.265		40.83% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
1.0	275	0.0750	4.41		**Shallow Concentrated Flow, Gravel ditch** Unpaved Kv= 16.1 fps
1.0	15	0.0200	0.24		**Sheet Flow, Road cross slope** Fallow n= 0.050 P2= 2.40"
2.0	290	Total			

Subcatchment 8S: Gravel & Paved Areas



Summary for Subcatchment 11S: Vegetated Area

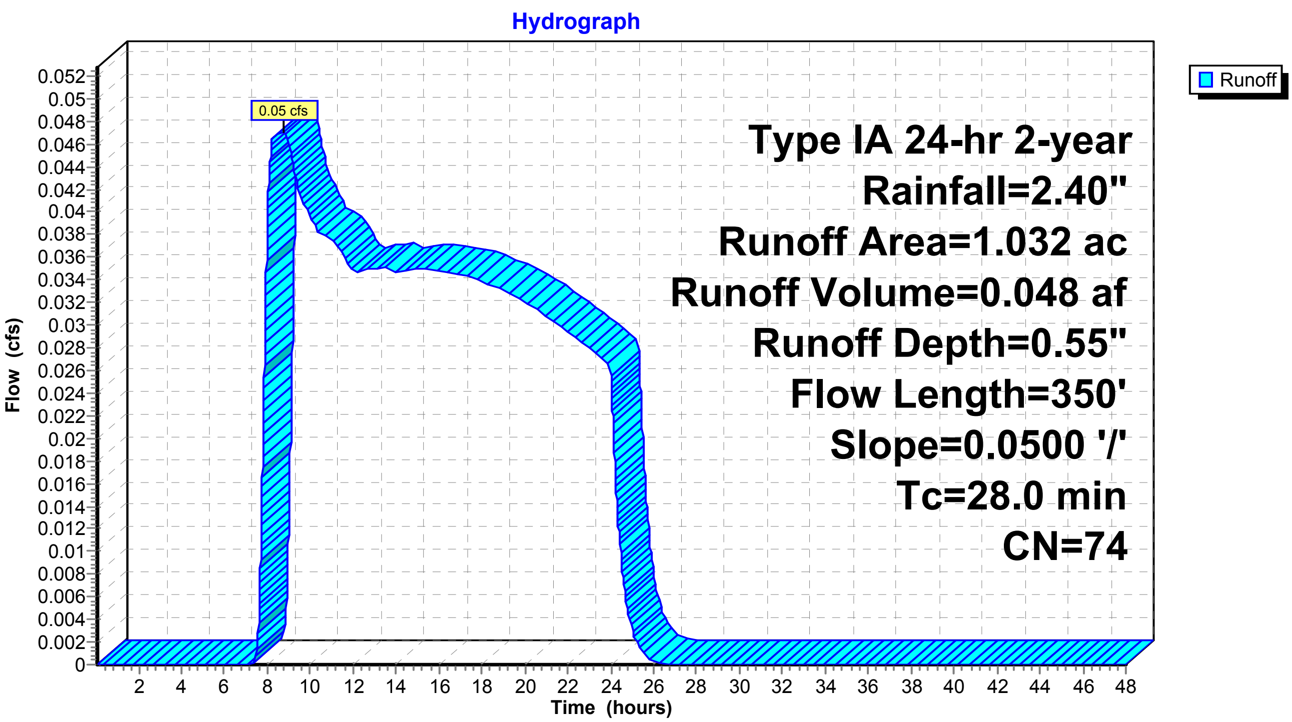
Runoff = 0.05 cfs @ 8.72 hrs, Volume= 0.048 af, Depth= 0.55"

Runoff by SBUH method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs
Type IA 24-hr 2-year Rainfall=2.40"

	Area (ac)	CN	Description
*	0.674	74	Grass, fair, HSG C
*	0.358	73	Woods, HSG C
	1.032	74	Weighted Average
	1.032		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
27.5	300	0.0500	0.18		**Sheet Flow, Upland area** Grass: Dense n= 0.240 P2= 2.40"
0.5	50	0.0500	1.57		**Shallow Concentrated Flow, Ditch** Short Grass Pasture Kv= 7.0 fps
28.0	350	Total			

Subcatchment 11S: Vegetated Area



Summary for Reach 11R: Swale

Inflow Area = 2.191 ac, 35.37% Impervious, Inflow Depth > 1.04" for 2-year event
Inflow = 0.08 cfs @ 24.00 hrs, Volume= 0.190 af
Outflow = 0.08 cfs @ 24.22 hrs, Volume= 0.189 af, Atten= 0%, Lag= 13.5 min

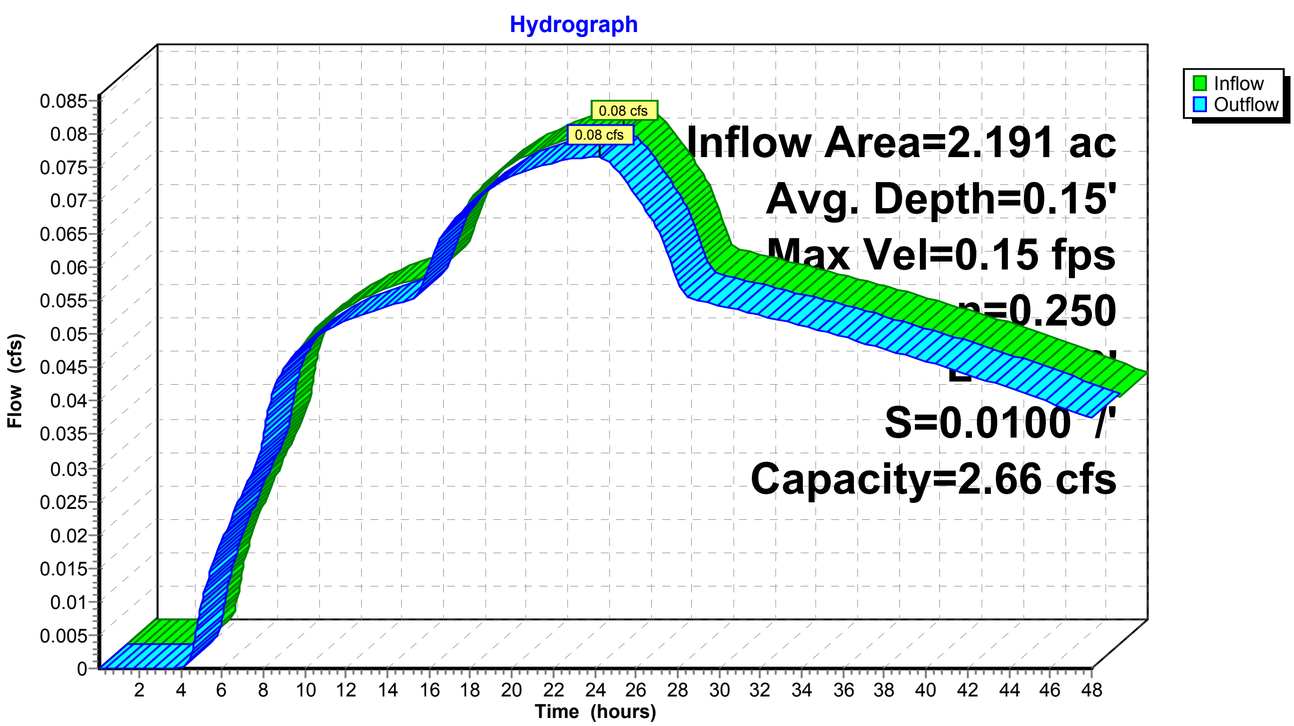
Routing by Stor-Ind+Trans method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs
Max. Velocity= 0.15 fps, Min. Travel Time= 12.1 min
Avg. Velocity = 0.13 fps, Avg. Travel Time= 14.1 min

Peak Storage= 56 cf @ 24.02 hrs, Average Depth at Peak Storage= 0.15'
Bank-Full Depth= 1.00', Capacity at Bank-Full= 2.66 cfs

3.00' x 1.00' deep channel, n= 0.250
Side Slope Z-value= 3.0 '/' Top Width= 9.00'
Length= 110.0' Slope= 0.0100 '/'
Inlet Invert= 426.00', Outlet Invert= 424.90'

‡

Reach 11R: Swale

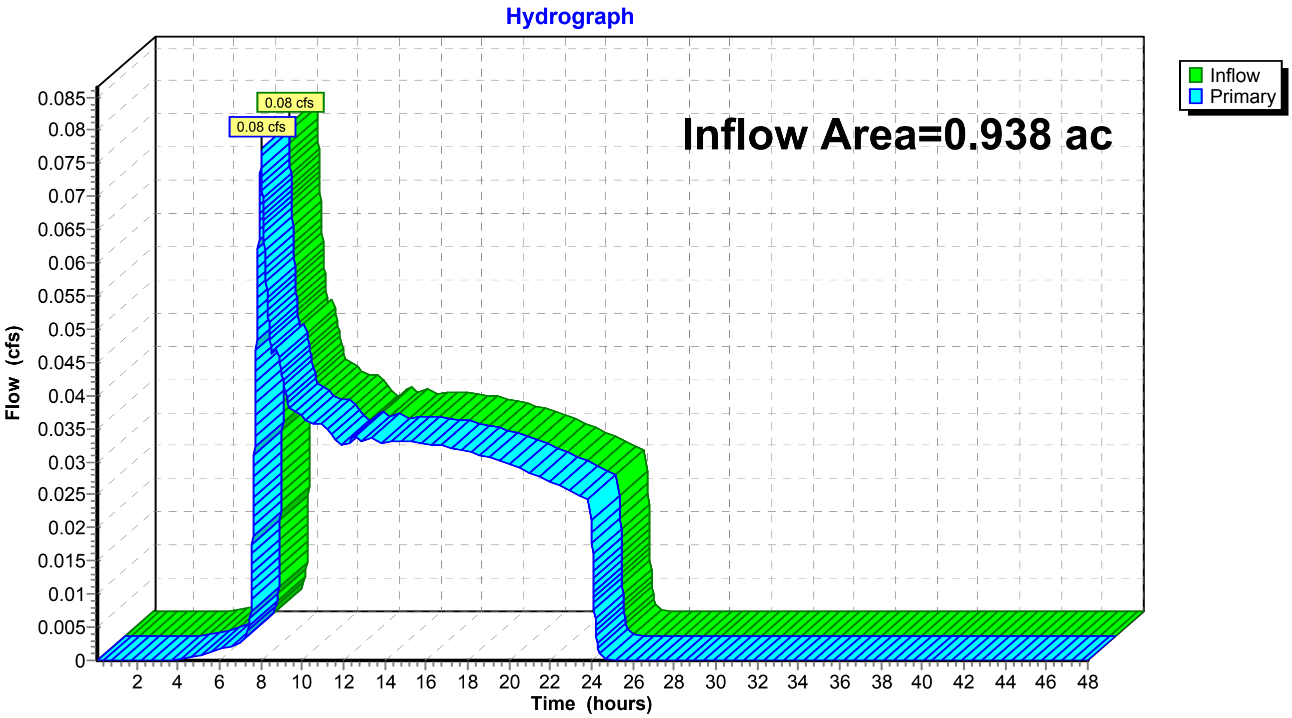


Summary for Pond 4P: Overland Runoff

Inflow Area =	0.938 ac,	0.00% Impervious, Inflow Depth = 0.59"	for 2-year event
Inflow =	0.08 cfs @	8.00 hrs, Volume=	0.046 af
Primary =	0.08 cfs @	8.00 hrs, Volume=	0.046 af, Atten= 0%, Lag= 0.0 min

Routing by Stor-Ind method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs

Pond 4P: Overland Runoff



Summary for Pond 9P: Pond Outlet

Inflow Area = 2.191 ac, 35.37% Impervious, Inflow Depth = 1.27" for 2-year event
Inflow = 0.60 cfs @ 7.89 hrs, Volume= 0.231 af
Outflow = 0.08 cfs @ 24.00 hrs, Volume= 0.190 af, Atten= 87%, Lag= 966.7 min
Primary = 0.08 cfs @ 24.00 hrs, Volume= 0.190 af

Routing by Stor-Ind method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs
Peak Elev= 435.51' @ 24.00 hrs Surf.Area= 0.044 ac Storage= 0.140 af

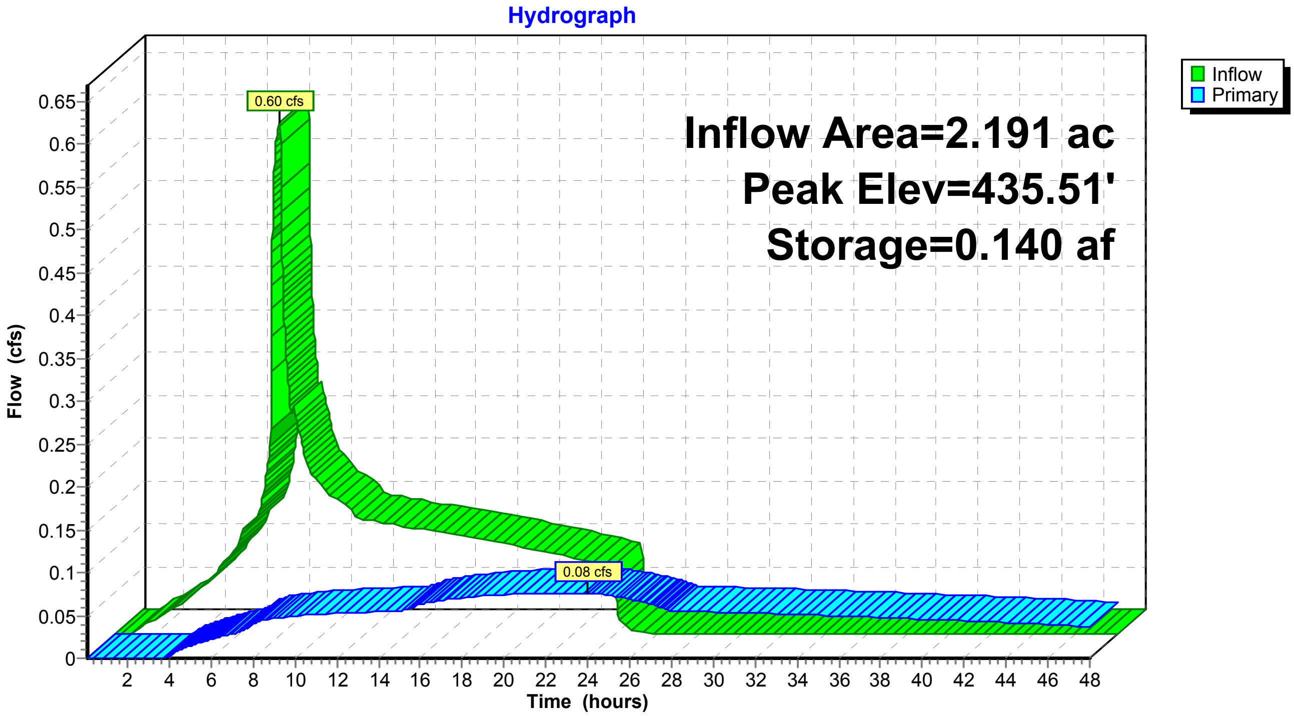
Plug-Flow detention time= 918.1 min calculated for 0.190 af (82% of inflow)
Center-of-Mass det. time= 803.9 min (1,554.5 - 750.6)

Volume	Invert	Avail.Storage	Storage Description
#1	430.00'	0.278 af	**15.00'W x 30.00'L x 8.00'H Prismatoid Z=2.0**

Device	Routing	Invert	Outlet Devices
#1	Primary	430.50'	**1.0" Vert. Orifice/Grate** C= 0.600
#2	Primary	435.00'	**1.0" Vert. Orifice/Grate** C= 0.600
#3	Primary	437.00'	**1.5" Vert. Orifice/Grate** C= 0.600
#4	Primary	438.00'	**45.0 deg x 3.0' long Sharp-Crested Vee/Trap Weir** C= 2.56

Primary OutFlow Max=0.08 cfs @ 24.00 hrs HW=435.51' (Free Discharge)
—**1=Orifice/Grate** (Orifice Controls 0.06 cfs @ 10.73 fps)
—**2=Orifice/Grate** (Orifice Controls 0.02 cfs @ 3.30 fps)
—**3=Orifice/Grate** (Controls 0.00 cfs)
—**4=Sharp-Crested Vee/Trap Weir** (Controls 0.00 cfs)

Pond 9P: Pond Outlet

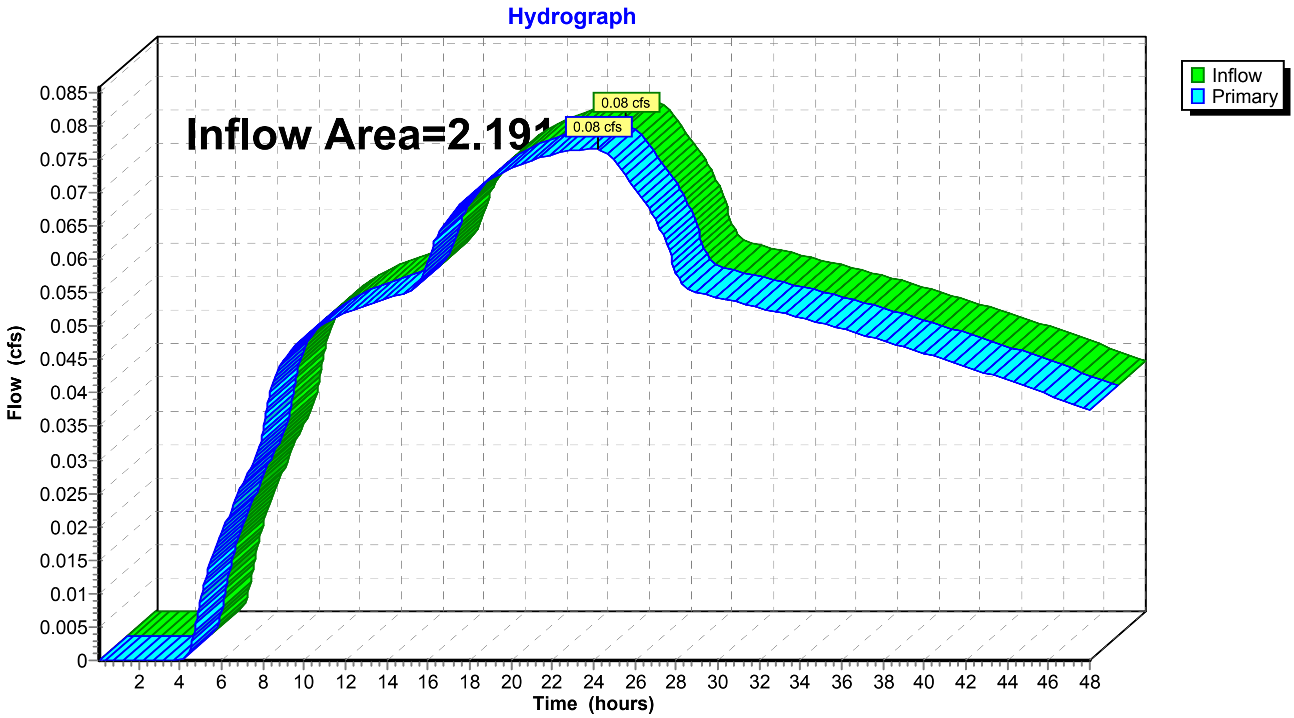


Summary for Pond 10P: Swale Outlet

Inflow Area =	2.191 ac, 35.37% Impervious, Inflow Depth > 1.03" for 2-year event	
Inflow =	0.08 cfs @ 24.22 hrs, Volume=	0.189 af
Primary =	0.08 cfs @ 24.22 hrs, Volume=	0.189 af, Atten= 0%, Lag= 0.0 min

Routing by Stor-Ind method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs

Pond 10P: Swale Outlet



Time span=0.08-48.00 hrs, dt=0.01 hrs, 4793 points
Runoff by SBUH method
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment2S: Grassy Area Runoff Area=0.897 ac 0.00% Impervious Runoff Depth=0.85"
Flow Length=110' Tc=7.0 min CN=74 Runoff=0.13 cfs 0.063 af

Subcatchment5S: Gravel Area Runoff Area=0.041 ac 0.00% Impervious Runoff Depth=1.81"
Flow Length=65' Tc=4.1 min CN=89 Runoff=0.02 cfs 0.006 af

Subcatchment6S: Reservoir Roof Runoff Area=0.510 ac 100.00% Impervious Runoff Depth=2.67"
Flow Length=360' Tc=2.2 min CN=98 Runoff=0.35 cfs 0.113 af

Subcatchment8S: Gravel & Paved Areas Runoff Area=0.649 ac 40.83% Impervious Runoff Depth=2.16"
Flow Length=290' Tc=2.0 min CN=93 Runoff=0.37 cfs 0.117 af

Subcatchment11S: Vegetated Area Runoff Area=1.032 ac 0.00% Impervious Runoff Depth=0.85"
Flow Length=350' Slope=0.0500 '/' Tc=28.0 min CN=74 Runoff=0.10 cfs 0.073 af

Reach 11R: Swale Avg. Depth=0.17' Max Vel=0.16 fps Inflow=0.09 cfs 0.241 af
n=0.250 L=110.0' S=0.0100 '/' Capacity=2.66 cfs Outflow=0.09 cfs 0.239 af

Pond 4P: Overland Runoff Inflow=0.15 cfs 0.069 af
Primary=0.15 cfs 0.069 af

Pond 9P: Pond Outlet Peak Elev=436.44' Storage=0.185 af Inflow=0.79 cfs 0.303 af
Outflow=0.09 cfs 0.241 af

Pond 10P: Swale Outlet Inflow=0.09 cfs 0.239 af
Primary=0.09 cfs 0.239 af

Total Runoff Area = 3.129 ac Runoff Volume = 0.372 af Average Runoff Depth = 1.43"
75.23% Pervious = 2.354 ac 24.77% Impervious = 0.775 ac

Summary for Subcatchment 2S: Grassy Area

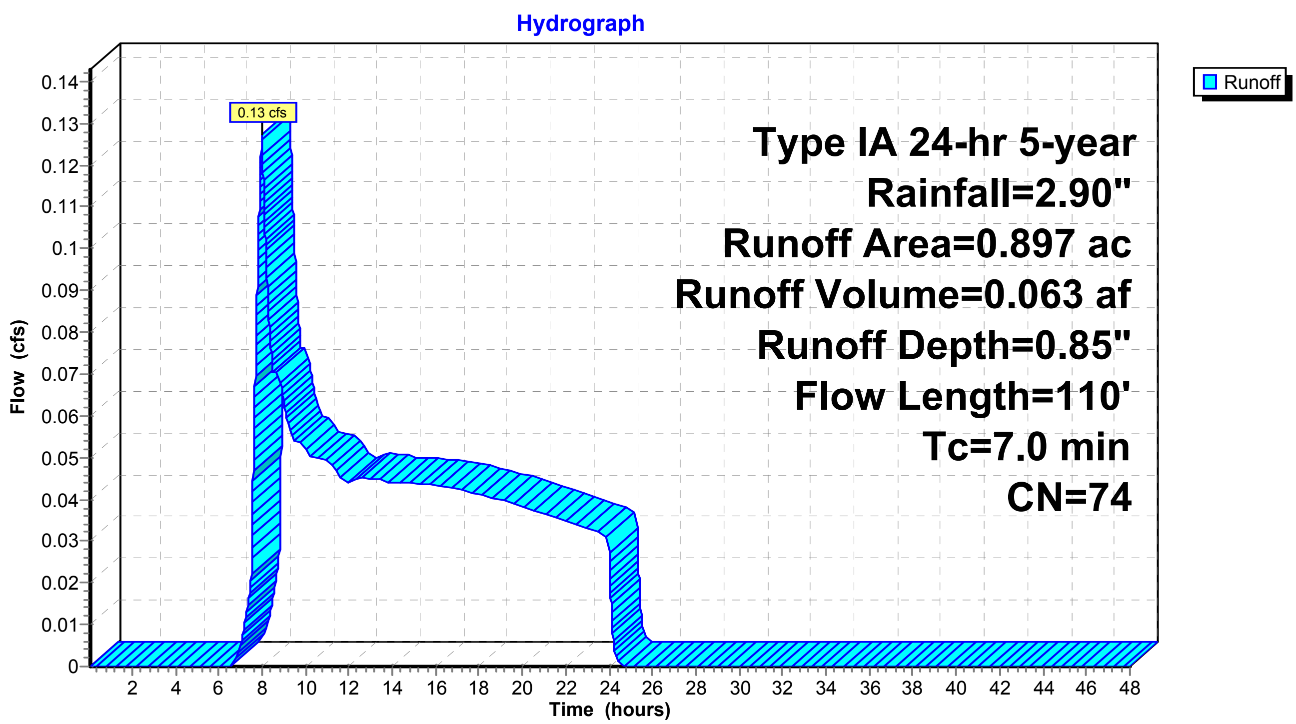
Runoff = 0.13 cfs @ 8.00 hrs, Volume= 0.063 af, Depth= 0.85"

Runoff by SBUH method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs
Type IA 24-hr 5-year Rainfall=2.90"

	Area (ac)	CN	Description
*	0.897	74	Grass, fair, HSG C
	0.897		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
3.6	60	0.3300	0.28		**Sheet Flow, steeper slope** Grass: Dense n= 0.240 P2= 2.40"
3.4	50	0.2500	0.24		**Sheet Flow, flatter slope** Grass: Dense n= 0.240 P2= 2.40"
7.0	110	Total			

Subcatchment 2S: Grassy Area



Summary for Subcatchment 5S: Gravel Area

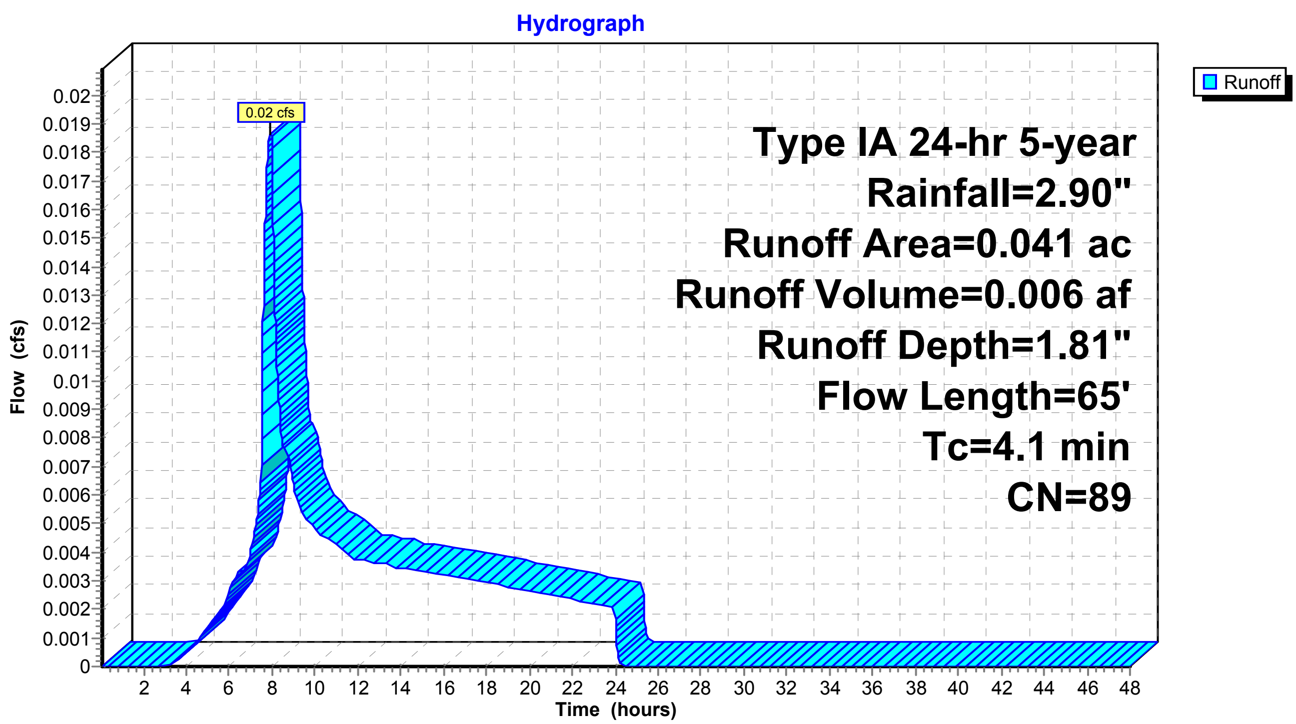
Runoff = 0.02 cfs @ 7.92 hrs, Volume= 0.006 af, Depth= 1.81"

Runoff by SBUH method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs
Type IA 24-hr 5-year Rainfall=2.90"

	Area (ac)	CN	Description
*	0.041	89	Gravel access to swale, HSG C
	0.041		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
1.0	15	0.0200	0.24		**Sheet Flow, Gravel access to pond** Fallow n= 0.050 P2= 2.40"
3.1	50	0.3300	0.27		**Sheet Flow, grass overland** Grass: Dense n= 0.240 P2= 2.40"
4.1	65	Total			

Subcatchment 5S: Gravel Area



Summary for Subcatchment 6S: Reservoir Roof

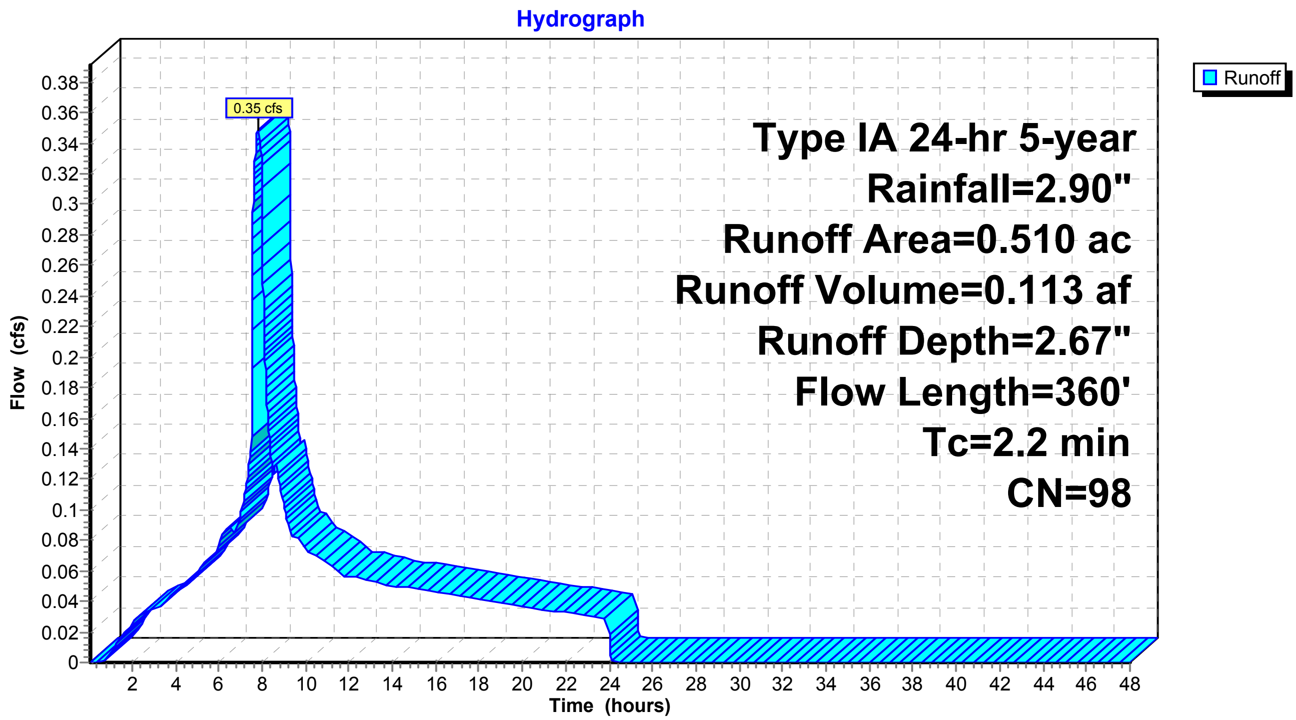
Runoff = 0.35 cfs @ 7.82 hrs, Volume= 0.113 af, Depth= 2.67"

Runoff by SBUH method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs
Type IA 24-hr 5-year Rainfall=2.90"

	Area (ac)	CN	Description
*	0.510	98	concrete roof
	0.510		100.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
1.2	85	0.0200	1.15		**Sheet Flow, Reservoir concrete roof** Smooth surfaces n= 0.011 P2= 2.40"
1.0	275	0.0750	4.41		**Shallow Concentrated Flow, gravel collection** Unpaved Kv= 16.1 fps
2.2	360	Total			

Subcatchment 6S: Reservoir Roof



Summary for Subcatchment 8S: Gravel & Paved Areas

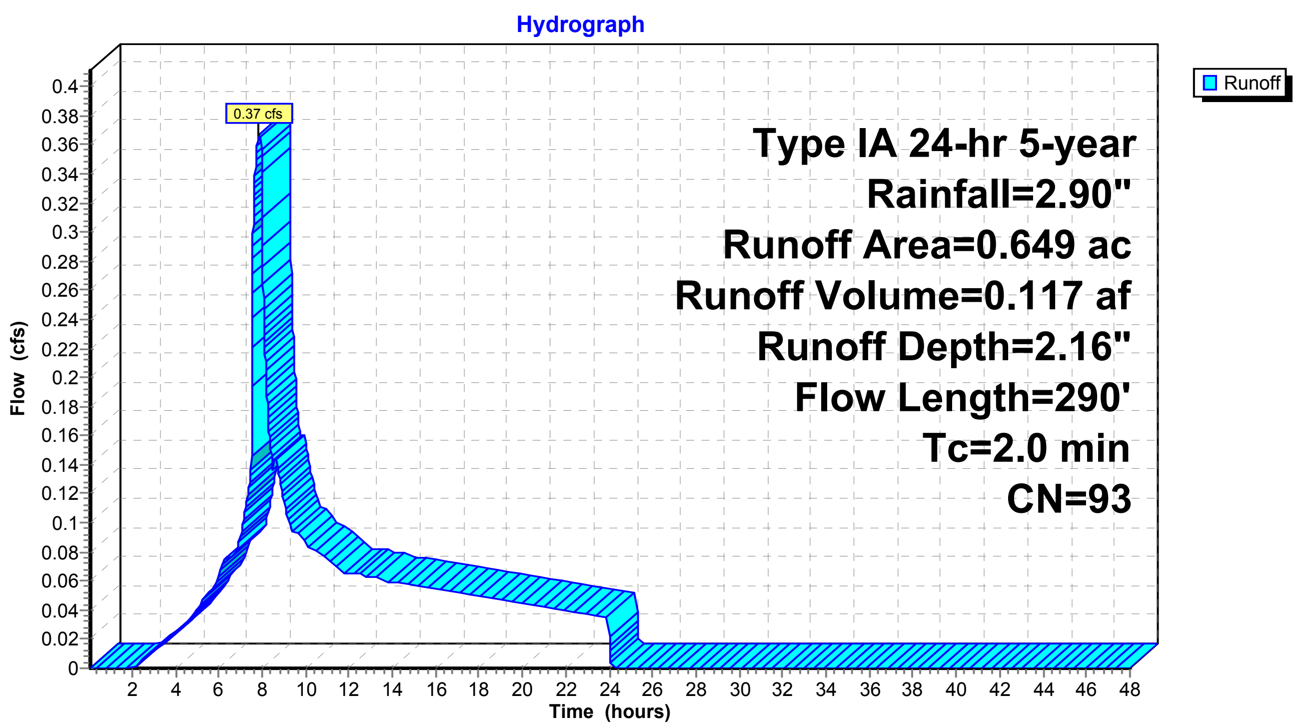
Runoff = 0.37 cfs @ 7.86 hrs, Volume= 0.117 af, Depth= 2.16"

Runoff by SBUH method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs
Type IA 24-hr 5-year Rainfall=2.90"

	Area (ac)	CN	Description
*	0.384	89	Gravel access
*	0.265	98	Paved access
	0.649	93	Weighted Average
	0.384		59.17% Pervious Area
	0.265		40.83% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
1.0	275	0.0750	4.41		**Shallow Concentrated Flow, Gravel ditch** Unpaved Kv= 16.1 fps
1.0	15	0.0200	0.24		**Sheet Flow, Road cross slope** Fallow n= 0.050 P2= 2.40"
2.0	290	Total			

Subcatchment 8S: Gravel & Paved Areas



Summary for Subcatchment 11S: Vegetated Area

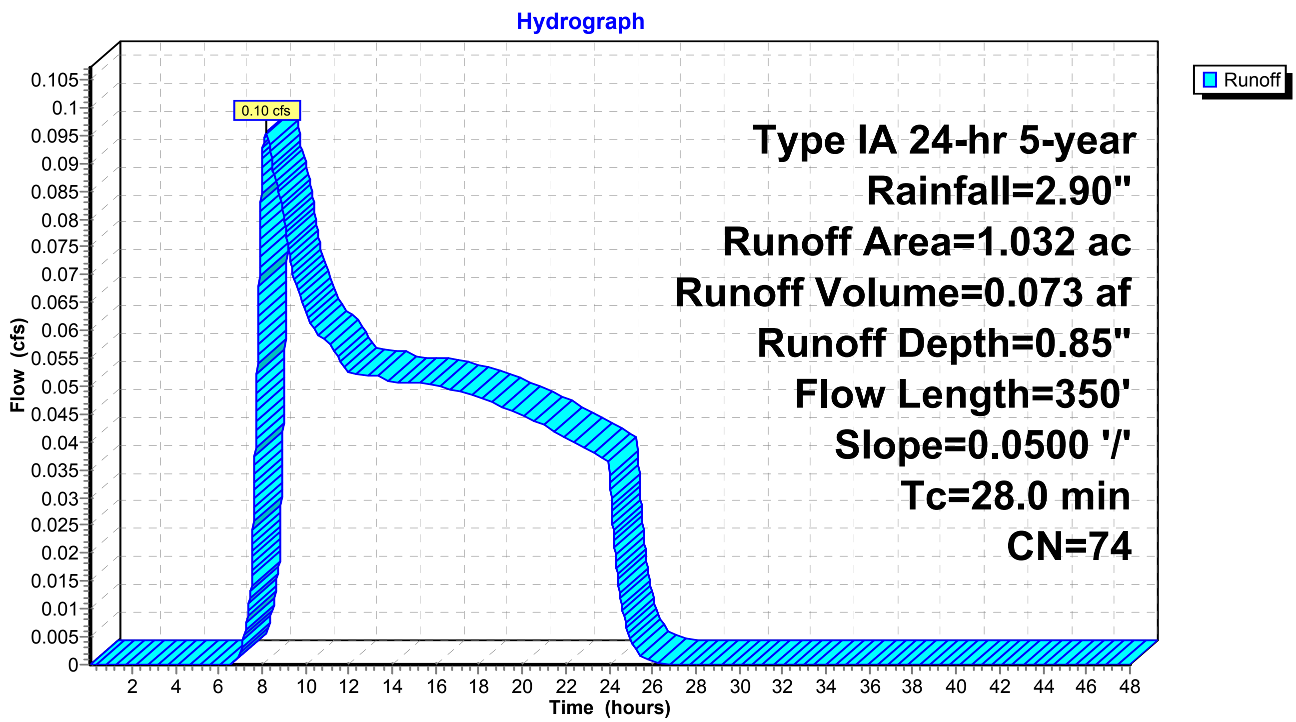
Runoff = 0.10 cfs @ 8.19 hrs, Volume= 0.073 af, Depth= 0.85"

Runoff by SBUH method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs
Type IA 24-hr 5-year Rainfall=2.90"

	Area (ac)	CN	Description
*	0.674	74	Grass, fair, HSG C
*	0.358	73	Woods, HSG C
	1.032	74	Weighted Average
	1.032		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
27.5	300	0.0500	0.18		**Sheet Flow, Upland area** Grass: Dense n= 0.240 P2= 2.40"
0.5	50	0.0500	1.57		**Shallow Concentrated Flow, Ditch** Short Grass Pasture Kv= 7.0 fps
28.0	350	Total			

Subcatchment 11S: Vegetated Area



Summary for Reach 11R: Swale

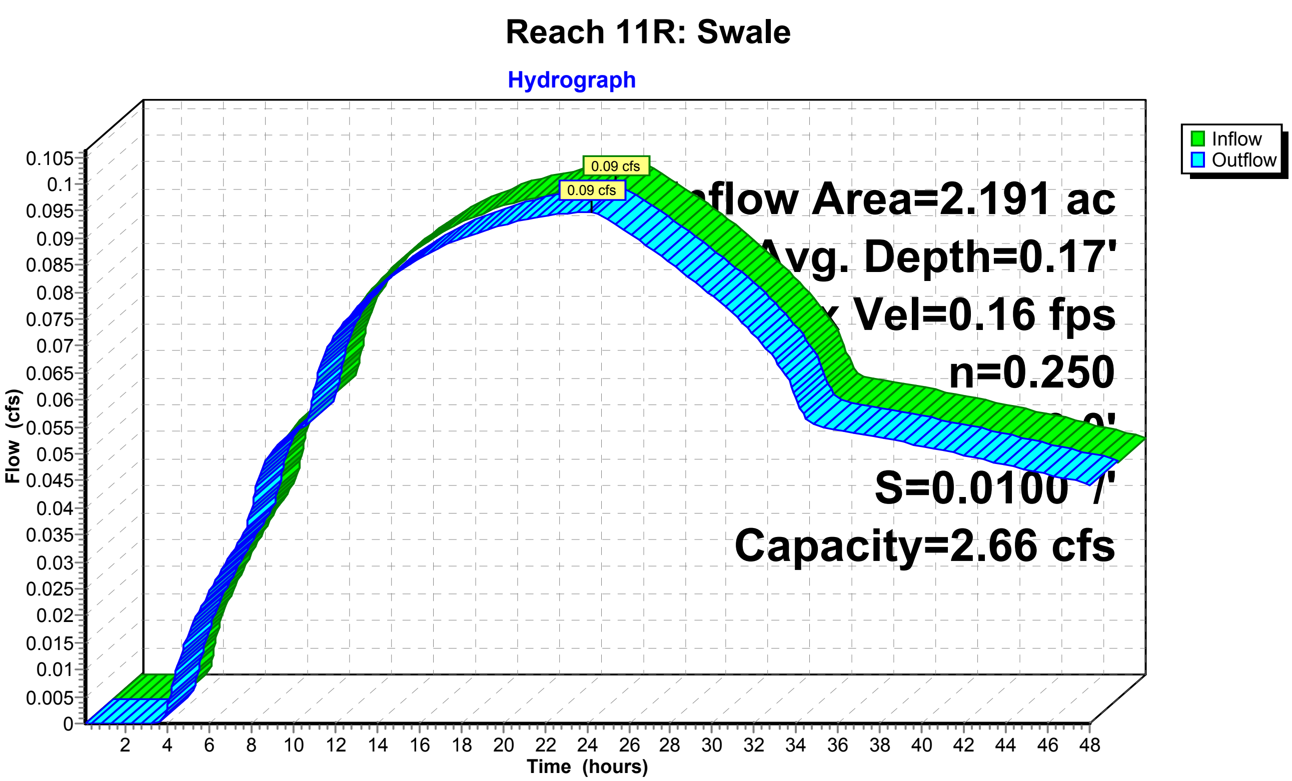
Inflow Area = 2.191 ac, 35.37% Impervious, Inflow Depth > 1.32" for 5-year event
Inflow = 0.09 cfs @ 24.00 hrs, Volume= 0.241 af
Outflow = 0.09 cfs @ 24.22 hrs, Volume= 0.239 af, Atten= 0%, Lag= 13.0 min

Routing by Stor-Ind+Trans method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs
Max. Velocity= 0.16 fps, Min. Travel Time= 11.2 min
Avg. Velocity = 0.14 fps, Avg. Travel Time= 13.1 min

Peak Storage= 64 cf @ 24.03 hrs, Average Depth at Peak Storage= 0.17'
Bank-Full Depth= 1.00', Capacity at Bank-Full= 2.66 cfs

3.00' x 1.00' deep channel, n= 0.250
Side Slope Z-value= 3.0 '/' Top Width= 9.00'
Length= 110.0' Slope= 0.0100 '/'
Inlet Invert= 426.00', Outlet Invert= 424.90'

‡

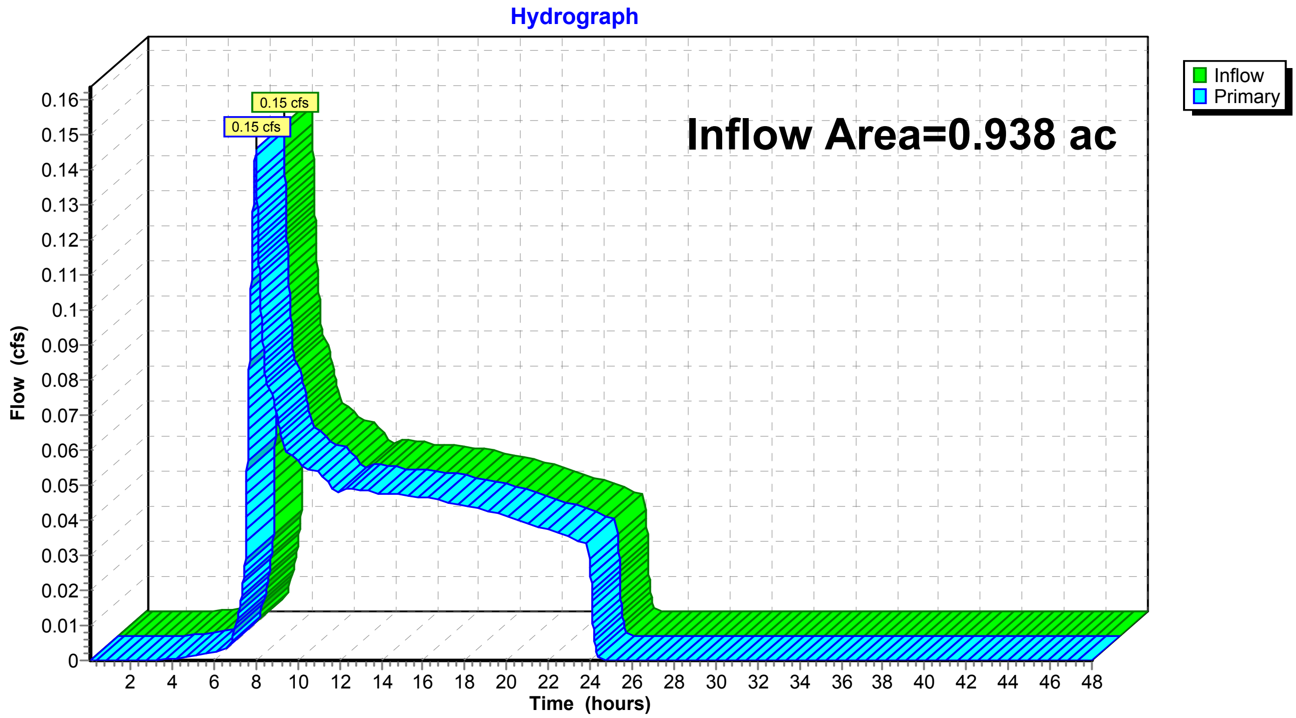


Summary for Pond 4P: Overland Runoff

Inflow Area =	0.938 ac,	0.00% Impervious,	Inflow Depth = 0.89"	for 5-year event	
Inflow =	0.15 cfs @	8.00 hrs,	Volume=	0.069 af	
Primary =	0.15 cfs @	8.00 hrs,	Volume=	0.069 af,	Atten= 0%, Lag= 0.0 min

Routing by Stor-Ind method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs

Pond 4P: Overland Runoff



Summary for Pond 9P: Pond Outlet

Inflow Area = 2.191 ac, 35.37% Impervious, Inflow Depth = 1.66" for 5-year event
Inflow = 0.79 cfs @ 7.89 hrs, Volume= 0.303 af
Outflow = 0.09 cfs @ 24.00 hrs, Volume= 0.241 af, Atten= 88%, Lag= 966.7 min
Primary = 0.09 cfs @ 24.00 hrs, Volume= 0.241 af

Routing by Stor-Ind method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs
Peak Elev= 436.44' @ 24.00 hrs Surf.Area= 0.052 ac Storage= 0.185 af

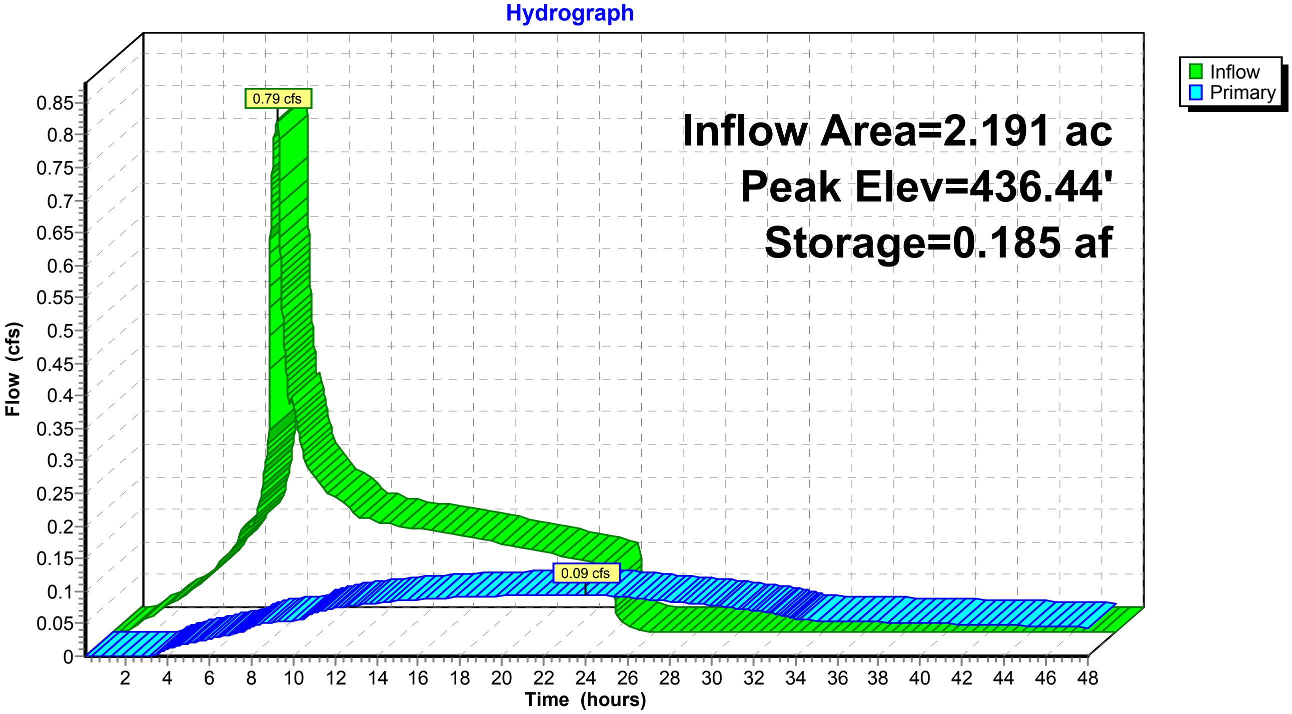
Plug-Flow detention time= 911.0 min calculated for 0.241 af (79% of inflow)
Center-of-Mass det. time= 780.3 min (1,524.7 - 744.4)

Volume	Invert	Avail.Storage	Storage Description
#1	430.00'	0.278 af	**15.00'W x 30.00'L x 8.00'H Prismatoid Z=2.0**

Device	Routing	Invert	Outlet Devices
#1	Primary	430.50'	**1.0" Vert. Orifice/Grate** C= 0.600
#2	Primary	435.00'	**1.0" Vert. Orifice/Grate** C= 0.600
#3	Primary	437.00'	**1.5" Vert. Orifice/Grate** C= 0.600
#4	Primary	438.00'	**45.0 deg x 3.0' long Sharp-Crested Vee/Trap Weir** C= 2.56

Primary OutFlow Max=0.09 cfs @ 24.00 hrs HW=436.44' (Free Discharge)
├─**1=Orifice/Grate** (Orifice Controls 0.06 cfs @ 11.69 fps)
├─**2=Orifice/Grate** (Orifice Controls 0.03 cfs @ 5.69 fps)
├─**3=Orifice/Grate** (Controls 0.00 cfs)
└─**4=Sharp-Crested Vee/Trap Weir** (Controls 0.00 cfs)

Pond 9P: Pond Outlet

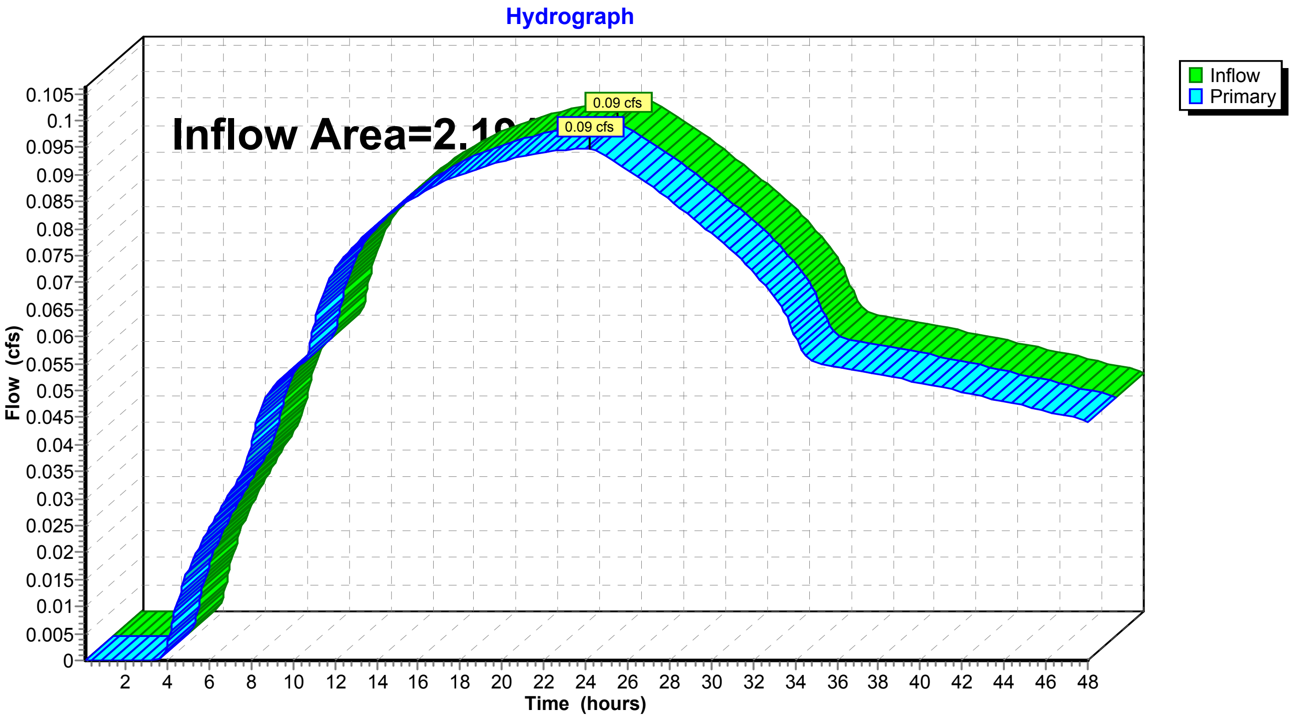


Summary for Pond 10P: Swale Outlet

Inflow Area =	2.191 ac, 35.37% Impervious, Inflow Depth > 1.31"	for 5-year event
Inflow =	0.09 cfs @ 24.22 hrs, Volume=	0.239 af
Primary =	0.09 cfs @ 24.22 hrs, Volume=	0.239 af, Atten= 0%, Lag= 0.0 min

Routing by Stor-Ind method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs

Pond 10P: Swale Outlet



Time span=0.08-48.00 hrs, dt=0.01 hrs, 4793 points
Runoff by SBUH method
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment2S: Grassy Area Runoff Area=0.897 ac 0.00% Impervious Runoff Depth=1.17"
Flow Length=110' Tc=7.0 min CN=74 Runoff=0.20 cfs 0.088 af

Subcatchment5S: Gravel Area Runoff Area=0.041 ac 0.00% Impervious Runoff Depth=2.26"
Flow Length=65' Tc=4.1 min CN=89 Runoff=0.02 cfs 0.008 af

Subcatchment6S: Reservoir Roof Runoff Area=0.510 ac 100.00% Impervious Runoff Depth=3.17"
Flow Length=360' Tc=2.2 min CN=98 Runoff=0.41 cfs 0.135 af

Subcatchment8S: Gravel & Paved Areas Runoff Area=0.649 ac 40.83% Impervious Runoff Depth=2.64"
Flow Length=290' Tc=2.0 min CN=93 Runoff=0.45 cfs 0.143 af

Subcatchment11S: Vegetated Area Runoff Area=1.032 ac 0.00% Impervious Runoff Depth=1.17"
Flow Length=350' Slope=0.0500 '/' Tc=28.0 min CN=74 Runoff=0.15 cfs 0.101 af

Reach 11R: Swale Avg. Depth=0.20' Max Vel=0.18 fps Inflow=0.13 cfs 0.296 af
n=0.250 L=110.0' S=0.0100 '/' Capacity=2.66 cfs Outflow=0.13 cfs 0.295 af

Pond 4P: Overland Runoff Inflow=0.22 cfs 0.095 af
Primary=0.22 cfs 0.095 af

Pond 9P: Pond Outlet Peak Elev=437.23' Storage=0.229 af Inflow=0.98 cfs 0.378 af
Outflow=0.13 cfs 0.296 af

Pond 10P: Swale Outlet Inflow=0.13 cfs 0.295 af
Primary=0.13 cfs 0.295 af

Total Runoff Area = 3.129 ac Runoff Volume = 0.473 af Average Runoff Depth = 1.82"
75.23% Pervious = 2.354 ac 24.77% Impervious = 0.775 ac

Summary for Subcatchment 2S: Grassy Area

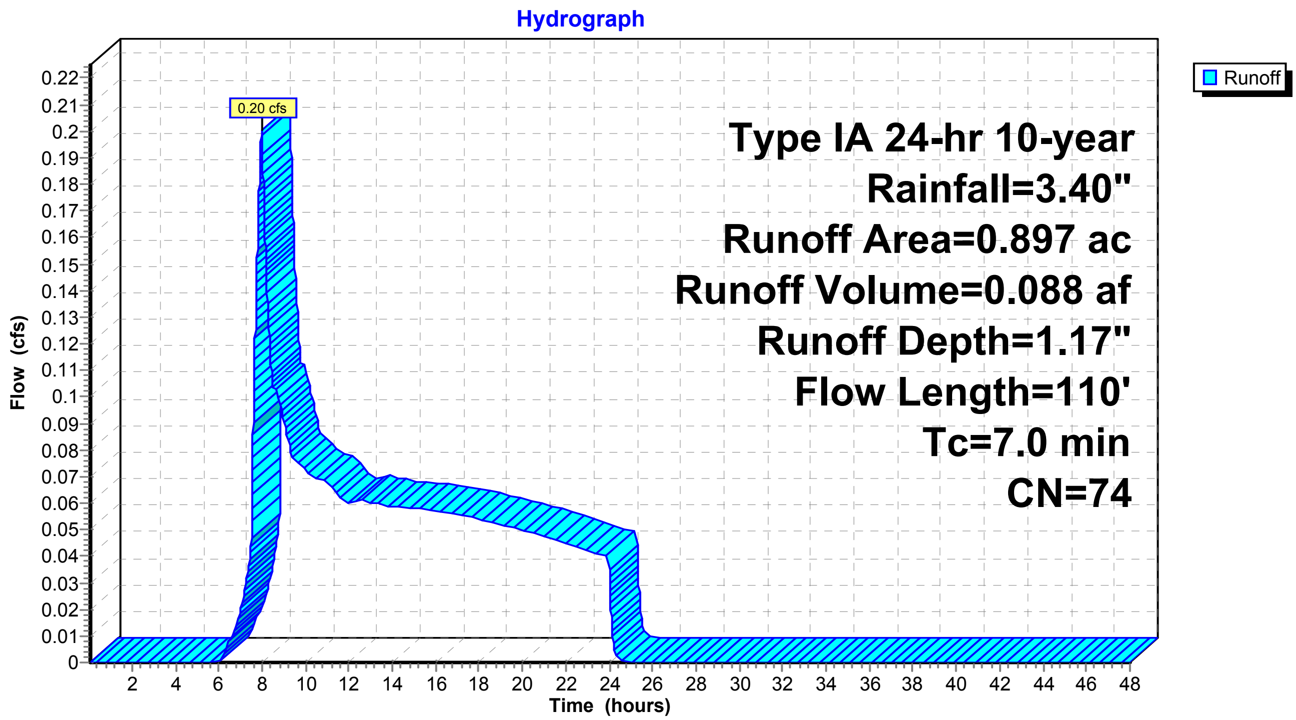
Runoff = 0.20 cfs @ 8.00 hrs, Volume= 0.088 af, Depth= 1.17"

Runoff by SBUH method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs
Type IA 24-hr 10-year Rainfall=3.40"

	Area (ac)	CN	Description
*	0.897	74	Grass, fair, HSG C
	0.897		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
3.6	60	0.3300	0.28		**Sheet Flow, steeper slope** Grass: Dense n= 0.240 P2= 2.40"
3.4	50	0.2500	0.24		**Sheet Flow, flatter slope** Grass: Dense n= 0.240 P2= 2.40"
7.0	110	Total			

Subcatchment 2S: Grassy Area



Summary for Subcatchment 5S: Gravel Area

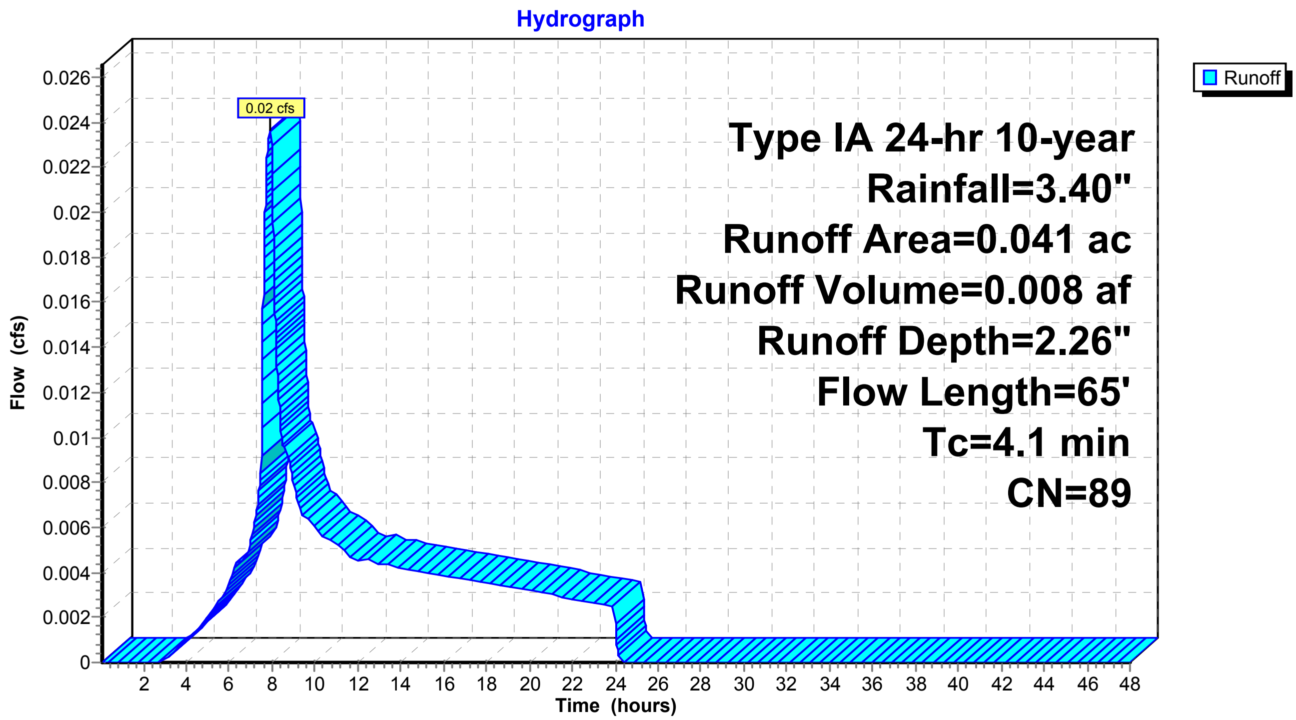
Runoff = 0.02 cfs @ 7.91 hrs, Volume= 0.008 af, Depth= 2.26"

Runoff by SBUH method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs
Type IA 24-hr 10-year Rainfall=3.40"

	Area (ac)	CN	Description
*	0.041	89	Gravel access to swale, HSG C
	0.041		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
1.0	15	0.0200	0.24		**Sheet Flow, Gravel access to pond** Fallow n= 0.050 P2= 2.40"
3.1	50	0.3300	0.27		**Sheet Flow, grass overland** Grass: Dense n= 0.240 P2= 2.40"
4.1	65	Total			

Subcatchment 5S: Gravel Area



Summary for Subcatchment 6S: Reservoir Roof

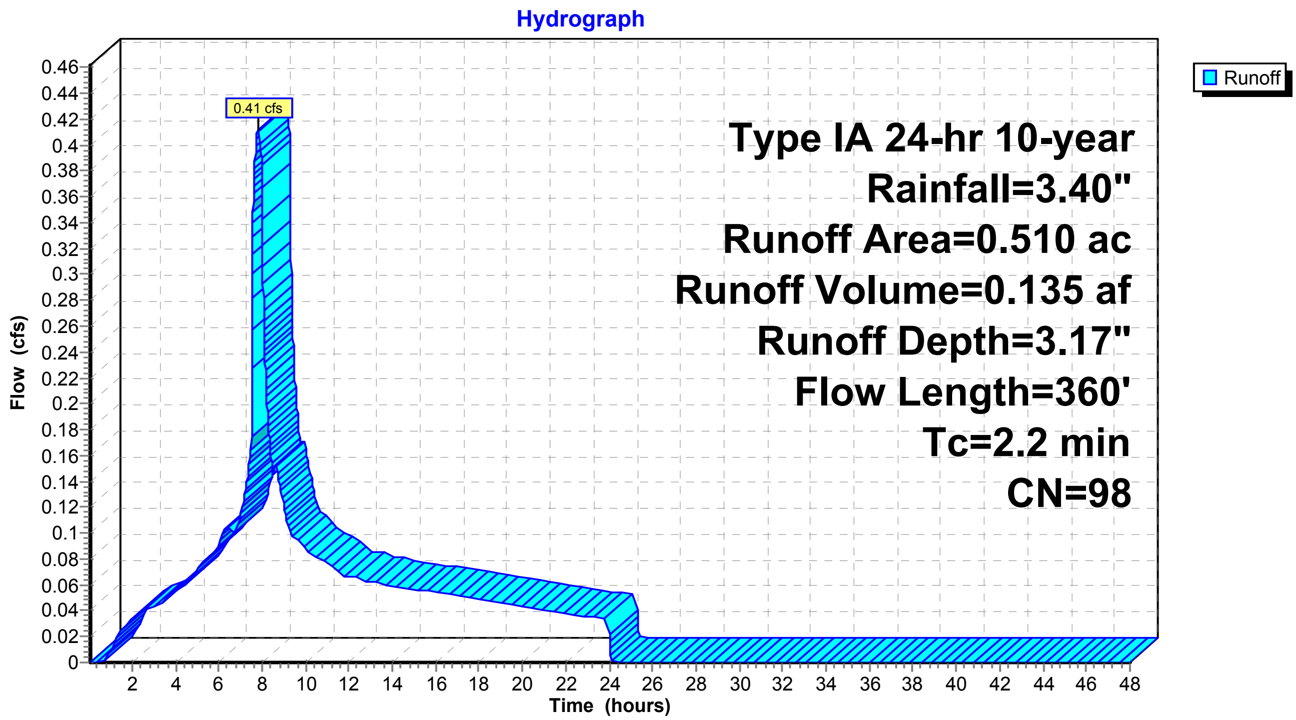
Runoff = 0.41 cfs @ 7.82 hrs, Volume= 0.135 af, Depth= 3.17"

Runoff by SBUH method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs
Type IA 24-hr 10-year Rainfall=3.40"

	Area (ac)	CN	Description
*	0.510	98	concrete roof
	0.510		100.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
1.2	85	0.0200	1.15		**Sheet Flow, Reservoir concrete roof** Smooth surfaces n= 0.011 P2= 2.40"
1.0	275	0.0750	4.41		**Shallow Concentrated Flow, gravel collection** Unpaved Kv= 16.1 fps
2.2	360	Total			

Subcatchment 6S: Reservoir Roof



Summary for Subcatchment 8S: Gravel & Paved Areas

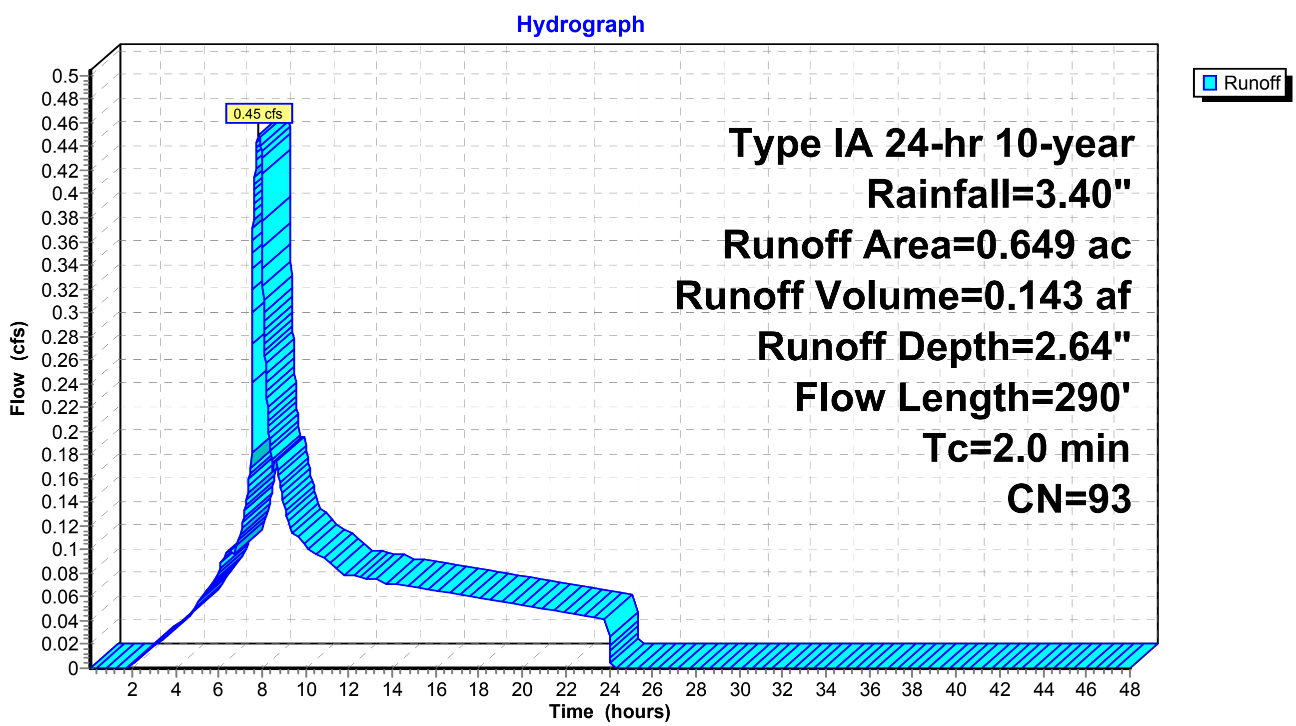
Runoff = 0.45 cfs @ 7.85 hrs, Volume= 0.143 af, Depth= 2.64"

Runoff by SBUH method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs
Type IA 24-hr 10-year Rainfall=3.40"

	Area (ac)	CN	Description
*	0.384	89	Gravel access
*	0.265	98	Paved access
	0.649	93	Weighted Average
	0.384		59.17% Pervious Area
	0.265		40.83% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
1.0	275	0.0750	4.41		**Shallow Concentrated Flow, Gravel ditch** Unpaved Kv= 16.1 fps
1.0	15	0.0200	0.24		**Sheet Flow, Road cross slope** Fallow n= 0.050 P2= 2.40"
2.0	290	Total			

Subcatchment 8S: Gravel & Paved Areas



Summary for Subcatchment 11S: Vegetated Area

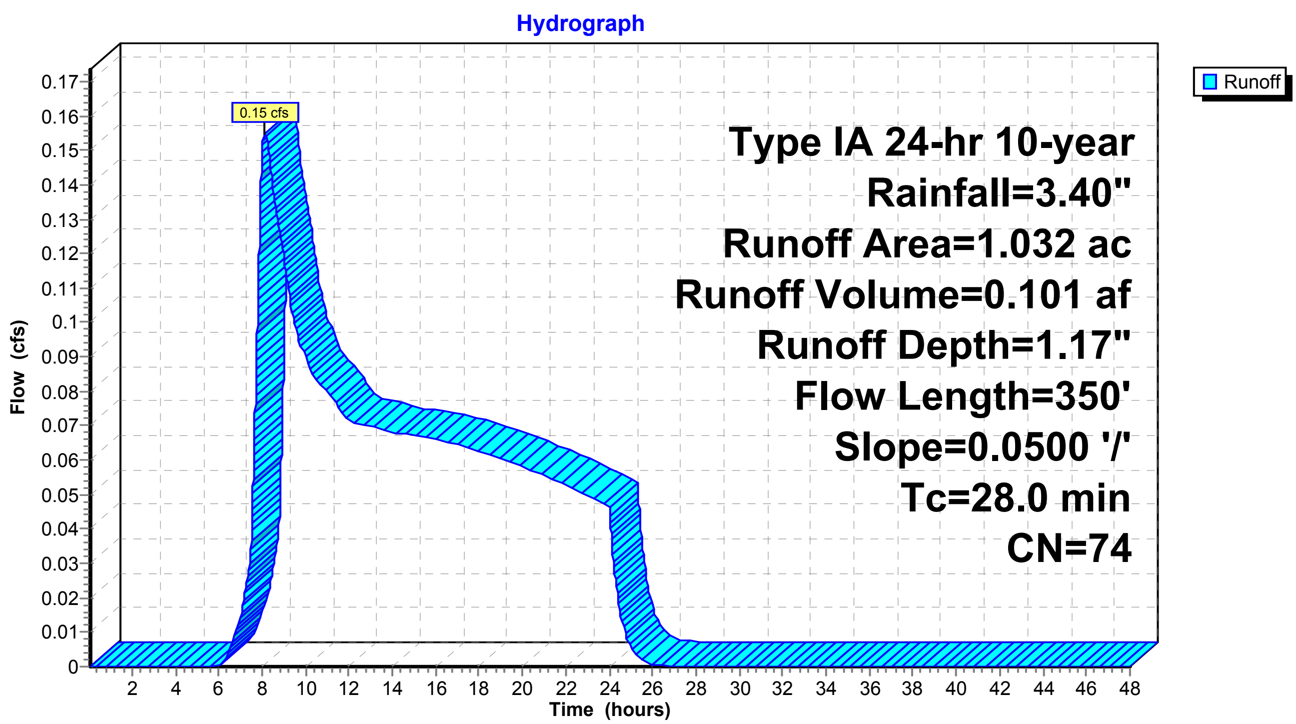
Runoff = 0.15 cfs @ 8.12 hrs, Volume= 0.101 af, Depth= 1.17"

Runoff by SBUH method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs
Type IA 24-hr 10-year Rainfall=3.40"

	Area (ac)	CN	Description
*	0.674	74	Grass, fair, HSG C
*	0.358	73	Woods, HSG C
	1.032	74	Weighted Average
	1.032		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
27.5	300	0.0500	0.18		**Sheet Flow, Upland area** Grass: Dense n= 0.240 P2= 2.40"
0.5	50	0.0500	1.57		**Shallow Concentrated Flow, Ditch** Short Grass Pasture Kv= 7.0 fps
28.0	350	Total			

Subcatchment 11S: Vegetated Area



Summary for Reach 11R: Swale

Inflow Area = 2.191 ac, 35.37% Impervious, Inflow Depth > 1.62" for 10-year event
Inflow = 0.13 cfs @ 22.80 hrs, Volume= 0.296 af
Outflow = 0.13 cfs @ 23.08 hrs, Volume= 0.295 af, Atten= 0%, Lag= 16.8 min

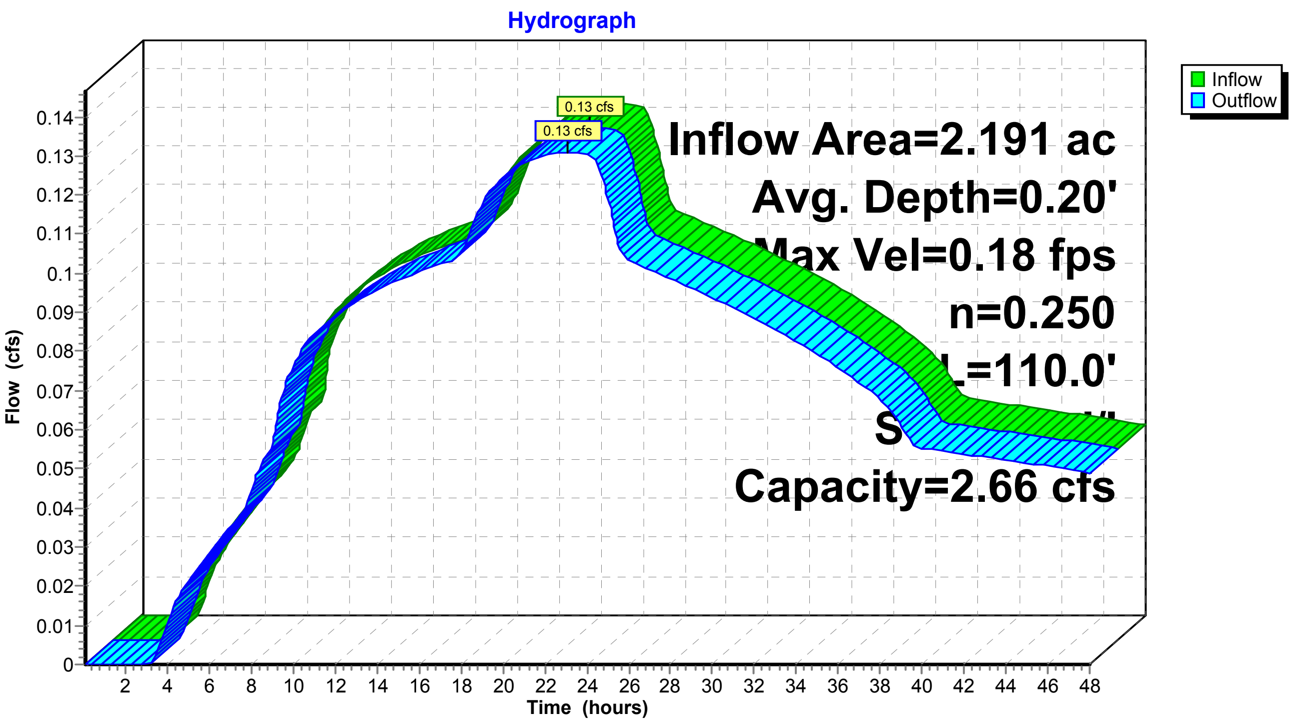
Routing by Stor-Ind+Trans method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs
Max. Velocity= 0.18 fps, Min. Travel Time= 10.1 min
Avg. Velocity = 0.15 fps, Avg. Travel Time= 12.3 min

Peak Storage= 79 cf @ 22.91 hrs, Average Depth at Peak Storage= 0.20'
Bank-Full Depth= 1.00', Capacity at Bank-Full= 2.66 cfs

3.00' x 1.00' deep channel, n= 0.250
Side Slope Z-value= 3.0 '/' Top Width= 9.00'
Length= 110.0' Slope= 0.0100 '/'
Inlet Invert= 426.00', Outlet Invert= 424.90'

‡

Reach 11R: Swale

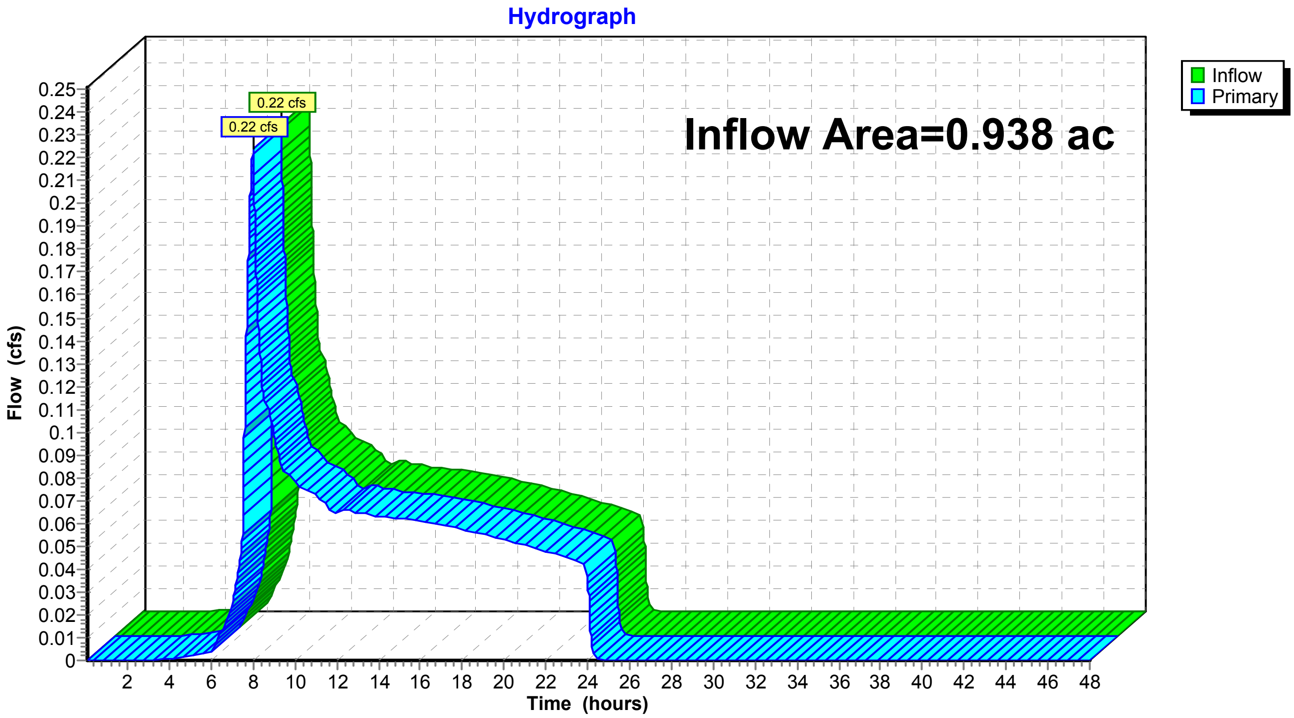


Summary for Pond 4P: Overland Runoff

Inflow Area =	0.938 ac,	0.00% Impervious, Inflow Depth = 1.22"	for 10-year event	
Inflow =	0.22 cfs @	8.00 hrs, Volume=	0.095 af	
Primary =	0.22 cfs @	8.00 hrs, Volume=	0.095 af,	Atten= 0%, Lag= 0.0 min

Routing by Stor-Ind method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs

Pond 4P: Overland Runoff



Summary for Pond 9P: Pond Outlet

Inflow Area = 2.191 ac, 35.37% Impervious, Inflow Depth = 2.07" for 10-year event
Inflow = 0.98 cfs @ 7.90 hrs, Volume= 0.378 af
Outflow = 0.13 cfs @ 22.80 hrs, Volume= 0.296 af, Atten= 87%, Lag= 894.0 min
Primary = 0.13 cfs @ 22.80 hrs, Volume= 0.296 af

Routing by Stor-Ind method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs
Peak Elev= 437.23' @ 22.80 hrs Surf.Area= 0.059 ac Storage= 0.229 af

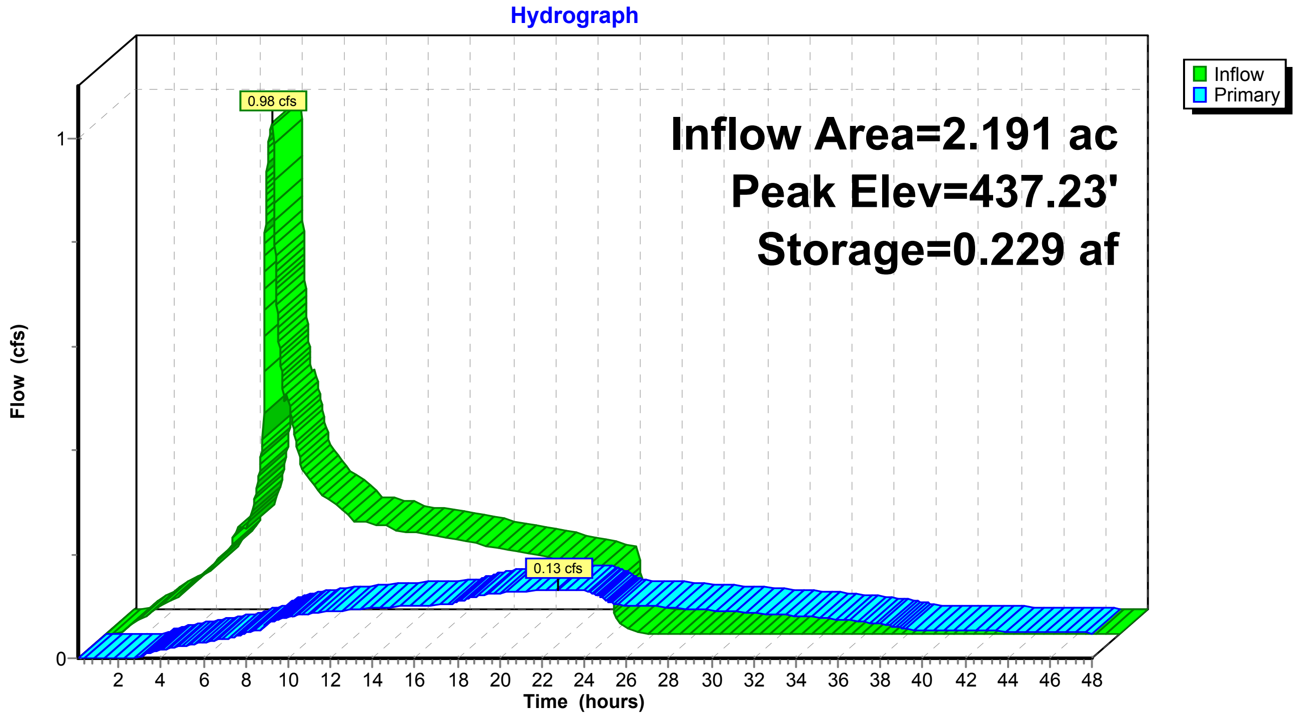
Plug-Flow detention time= 907.1 min calculated for 0.296 af (78% of inflow)
Center-of-Mass det. time= 769.7 min (1,508.5 - 738.9)

Volume	Invert	Avail.Storage	Storage Description
#1	430.00'	0.278 af	**15.00'W x 30.00'L x 8.00'H Prismatoid Z=2.0**

Device	Routing	Invert	Outlet Devices
#1	Primary	430.50'	**1.0" Vert. Orifice/Grate** C= 0.600
#2	Primary	435.00'	**1.0" Vert. Orifice/Grate** C= 0.600
#3	Primary	437.00'	**1.5" Vert. Orifice/Grate** C= 0.600
#4	Primary	438.00'	**45.0 deg x 3.0' long Sharp-Crested Vee/Trap Weir** C= 2.56

Primary OutFlow Max=0.13 cfs @ 22.80 hrs HW=437.23' (Free Discharge)
├─**1=Orifice/Grate** (Orifice Controls 0.07 cfs @ 12.45 fps)
├─**2=Orifice/Grate** (Orifice Controls 0.04 cfs @ 7.12 fps)
├─**3=Orifice/Grate** (Orifice Controls 0.02 cfs @ 1.97 fps)
└─**4=Sharp-Crested Vee/Trap Weir** (Controls 0.00 cfs)

Pond 9P: Pond Outlet

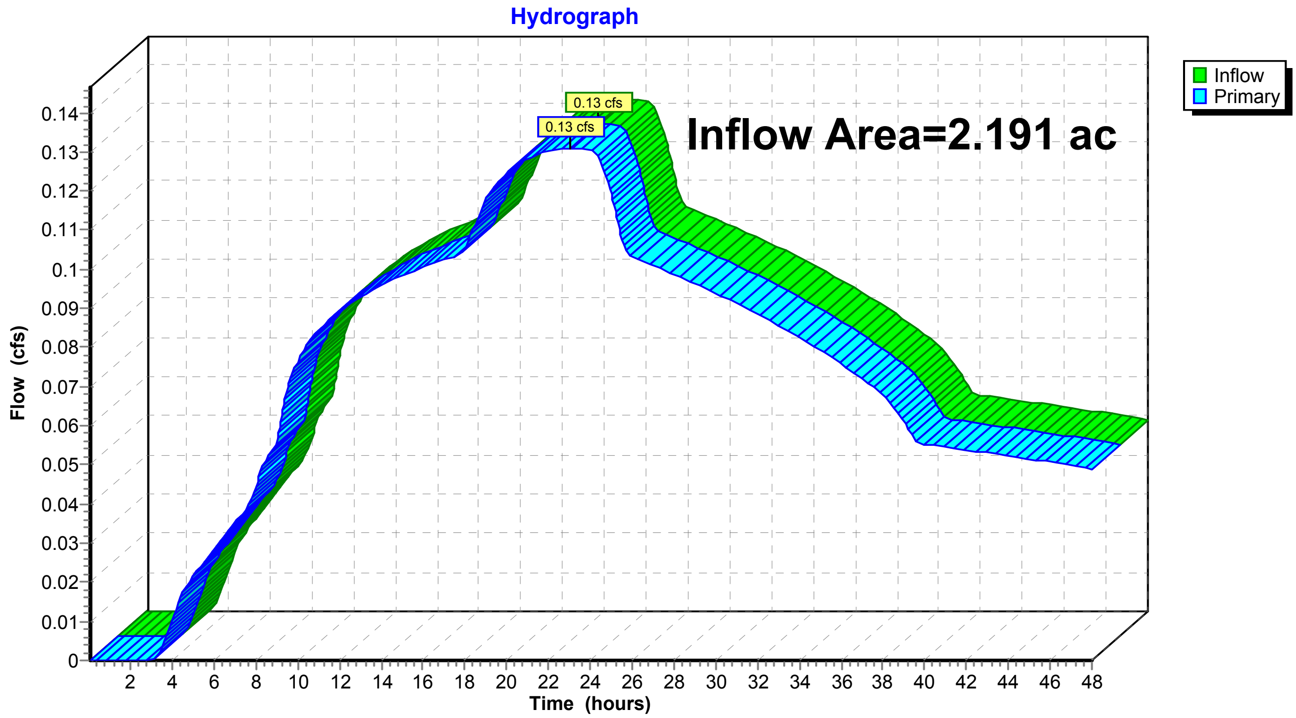


Summary for Pond 10P: Swale Outlet

Inflow Area = 2.191 ac, 35.37% Impervious, Inflow Depth > 1.61" for 10-year event
Inflow = 0.13 cfs @ 23.08 hrs, Volume= 0.295 af
Primary = 0.13 cfs @ 23.08 hrs, Volume= 0.295 af, Atten= 0%, Lag= 0.0 min

Routing by Stor-Ind method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs

Pond 10P: Swale Outlet



Time span=0.08-48.00 hrs, dt=0.01 hrs, 4793 points
Runoff by SBUH method
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment2S: Grassy Area Runoff Area=0.897 ac 0.00% Impervious Runoff Depth=1.52"
Flow Length=110' Tc=7.0 min CN=74 Runoff=0.28 cfs 0.114 af

Subcatchment5S: Gravel Area Runoff Area=0.041 ac 0.00% Impervious Runoff Depth=2.73"
Flow Length=65' Tc=4.1 min CN=89 Runoff=0.03 cfs 0.009 af

Subcatchment6S: Reservoir Roof Runoff Area=0.510 ac 100.00% Impervious Runoff Depth=3.67"
Flow Length=360' Tc=2.2 min CN=98 Runoff=0.47 cfs 0.156 af

Subcatchment8S: Gravel & Paved Areas Runoff Area=0.649 ac 40.83% Impervious Runoff Depth=3.12"
Flow Length=290' Tc=2.0 min CN=93 Runoff=0.53 cfs 0.169 af

Subcatchment11S: Vegetated Area Runoff Area=1.032 ac 0.00% Impervious Runoff Depth=1.52"
Flow Length=350' Slope=0.0500 '/' Tc=28.0 min CN=74 Runoff=0.22 cfs 0.131 af

Reach 11R: Swale Avg. Depth=0.23' Max Vel=0.20 fps Inflow=0.16 cfs 0.363 af
n=0.250 L=110.0' S=0.0100 '/' Capacity=2.66 cfs Outflow=0.16 cfs 0.362 af

Pond 4P: Overland Runoff Inflow=0.31 cfs 0.123 af
Primary=0.31 cfs 0.123 af

Pond 9P: Pond Outlet Peak Elev=437.78' Storage=0.263 af Inflow=1.19 cfs 0.456 af
Outflow=0.16 cfs 0.363 af

Pond 10P: Swale Outlet Inflow=0.16 cfs 0.362 af
Primary=0.16 cfs 0.362 af

Total Runoff Area = 3.129 ac Runoff Volume = 0.579 af Average Runoff Depth = 2.22"
75.23% Pervious = 2.354 ac 24.77% Impervious = 0.775 ac

Summary for Subcatchment 2S: Grassy Area

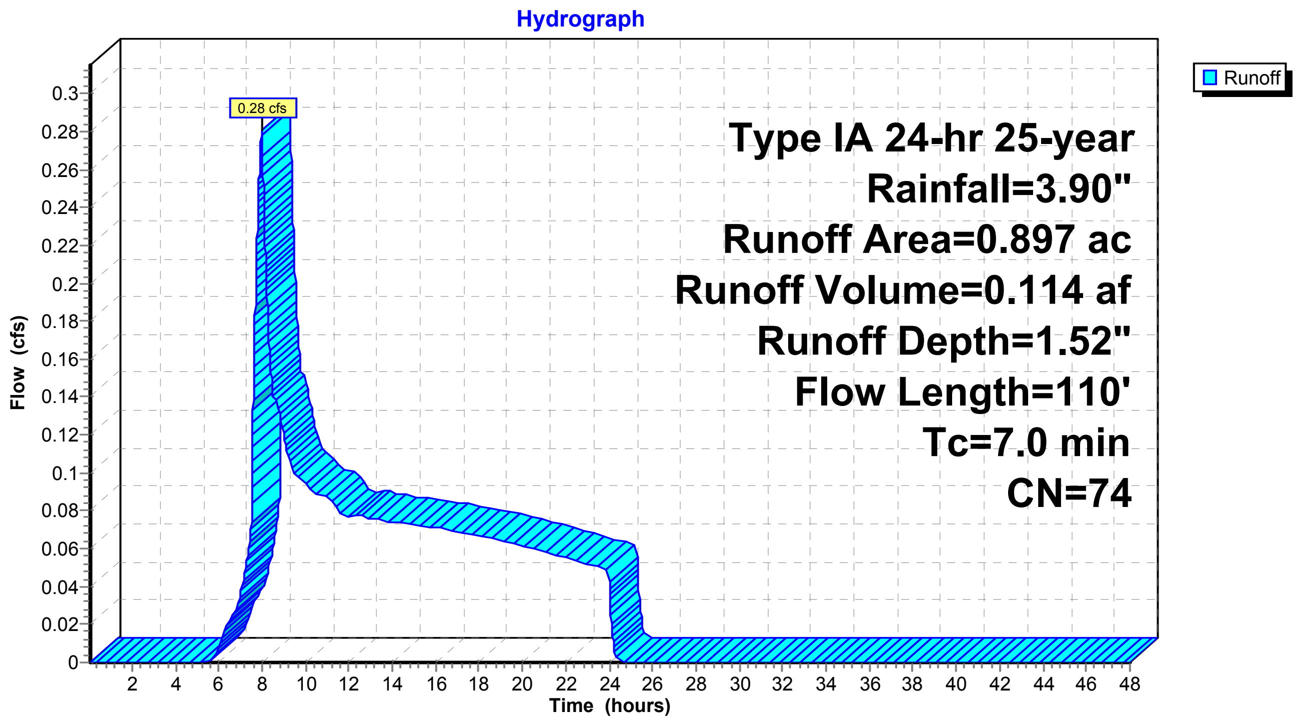
Runoff = 0.28 cfs @ 8.00 hrs, Volume= 0.114 af, Depth= 1.52"

Runoff by SBUH method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs
Type IA 24-hr 25-year Rainfall=3.90"

	Area (ac)	CN	Description
*	0.897	74	Grass, fair, HSG C
	0.897		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
3.6	60	0.3300	0.28		**Sheet Flow, steeper slope** Grass: Dense n= 0.240 P2= 2.40"
3.4	50	0.2500	0.24		**Sheet Flow, flatter slope** Grass: Dense n= 0.240 P2= 2.40"
7.0	110	Total			

Subcatchment 2S: Grassy Area



Summary for Subcatchment 5S: Gravel Area

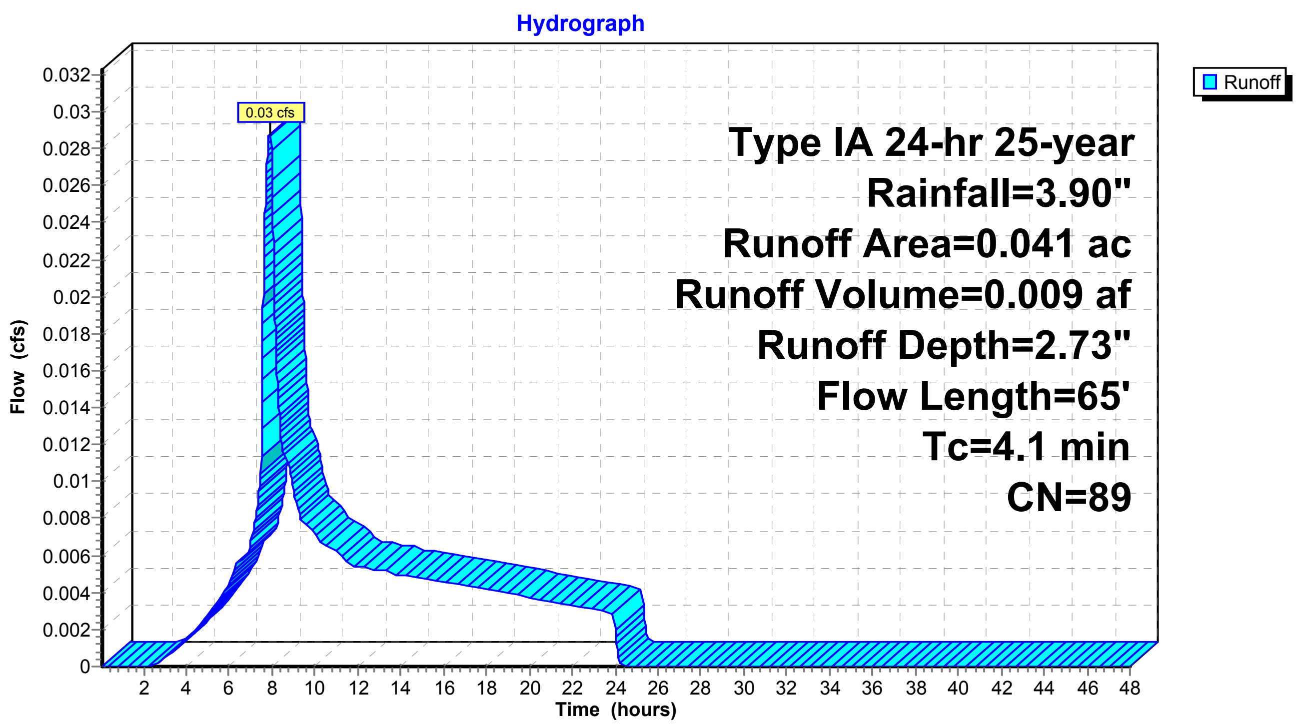
Runoff = 0.03 cfs @ 7.90 hrs, Volume= 0.009 af, Depth= 2.73"

Runoff by SBUH method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs
Type IA 24-hr 25-year Rainfall=3.90"

	Area (ac)	CN	Description
*	0.041	89	Gravel access to swale, HSG C
	0.041		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
1.0	15	0.0200	0.24		**Sheet Flow, Gravel access to pond** Fallow n= 0.050 P2= 2.40"
3.1	50	0.3300	0.27		**Sheet Flow, grass overland** Grass: Dense n= 0.240 P2= 2.40"
4.1	65	Total			

Subcatchment 5S: Gravel Area



Summary for Subcatchment 6S: Reservoir Roof

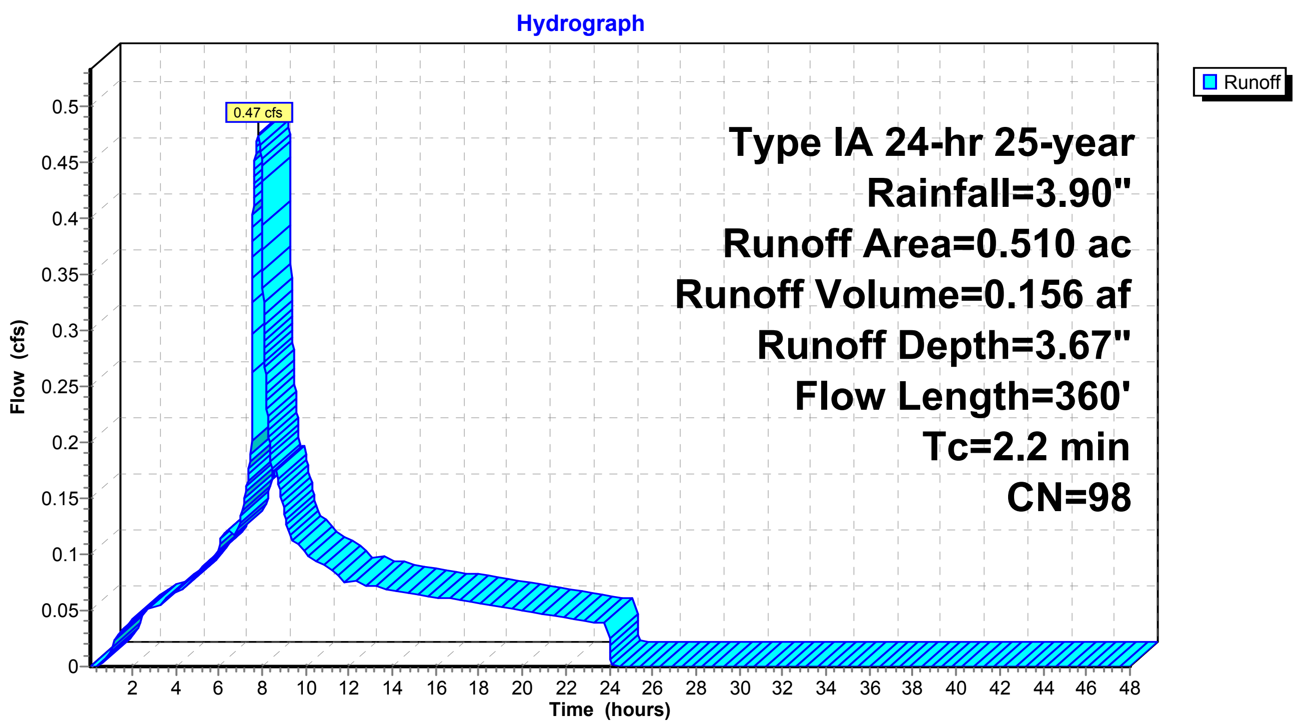
Runoff = 0.47 cfs @ 7.82 hrs, Volume= 0.156 af, Depth= 3.67"

Runoff by SBUH method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs
Type IA 24-hr 25-year Rainfall=3.90"

	Area (ac)	CN	Description
*	0.510	98	concrete roof
	0.510		100.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
1.2	85	0.0200	1.15		**Sheet Flow, Reservoir concrete roof** Smooth surfaces n= 0.011 P2= 2.40"
1.0	275	0.0750	4.41		**Shallow Concentrated Flow, gravel collection** Unpaved Kv= 16.1 fps
2.2	360	Total			

Subcatchment 6S: Reservoir Roof



Summary for Subcatchment 8S: Gravel & Paved Areas

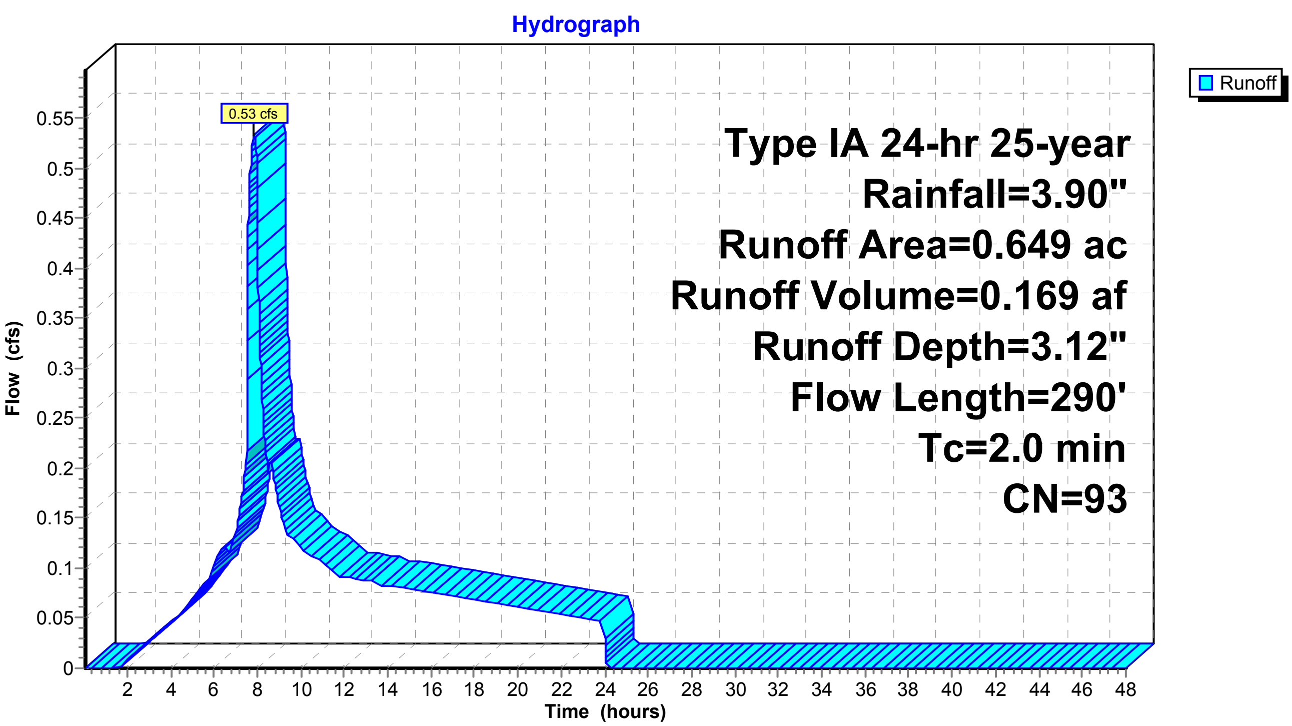
Runoff = 0.53 cfs @ 7.84 hrs, Volume= 0.169 af, Depth= 3.12"

Runoff by SBUH method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs
Type IA 24-hr 25-year Rainfall=3.90"

	Area (ac)	CN	Description
*	0.384	89	Gravel access
*	0.265	98	Paved access
	0.649	93	Weighted Average
	0.384		59.17% Pervious Area
	0.265		40.83% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
1.0	275	0.0750	4.41		**Shallow Concentrated Flow, Gravel ditch** Unpaved Kv= 16.1 fps
1.0	15	0.0200	0.24		**Sheet Flow, Road cross slope** Fallow n= 0.050 P2= 2.40"
2.0	290	Total			

Subcatchment 8S: Gravel & Paved Areas



Summary for Subcatchment 11S: Vegetated Area

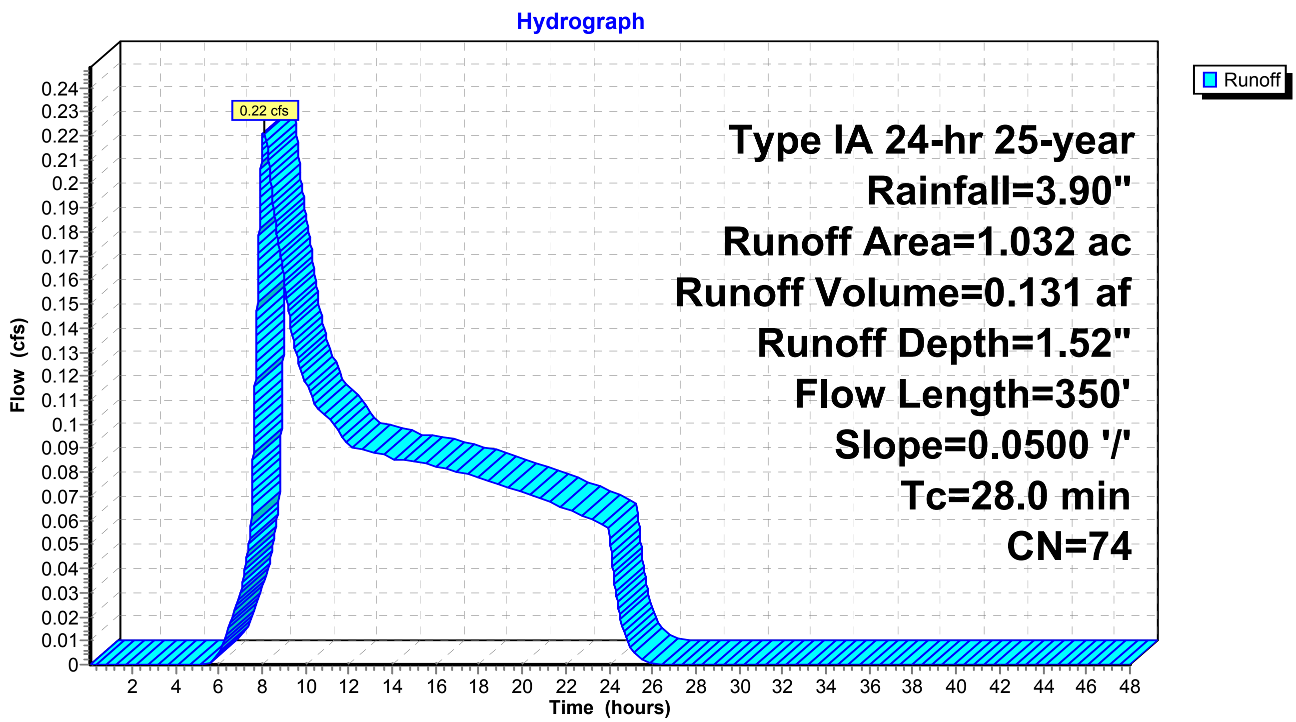
Runoff = 0.22 cfs @ 8.07 hrs, Volume= 0.131 af, Depth= 1.52"

Runoff by SBUH method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs
Type IA 24-hr 25-year Rainfall=3.90"

	Area (ac)	CN	Description
*	0.674	74	Grass, fair, HSG C
*	0.358	73	Woods, HSG C
	1.032	74	Weighted Average
	1.032		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
27.5	300	0.0500	0.18		**Sheet Flow, Upland area** Grass: Dense n= 0.240 P2= 2.40"
0.5	50	0.0500	1.57		**Shallow Concentrated Flow, Ditch** Short Grass Pasture Kv= 7.0 fps
28.0	350	Total			

Subcatchment 11S: Vegetated Area



Summary for Reach 11R: Swale

Inflow Area = 2.191 ac, 35.37% Impervious, Inflow Depth > 1.99" for 25-year event
Inflow = 0.16 cfs @ 21.90 hrs, Volume= 0.363 af
Outflow = 0.16 cfs @ 22.16 hrs, Volume= 0.362 af, Atten= 0%, Lag= 15.7 min

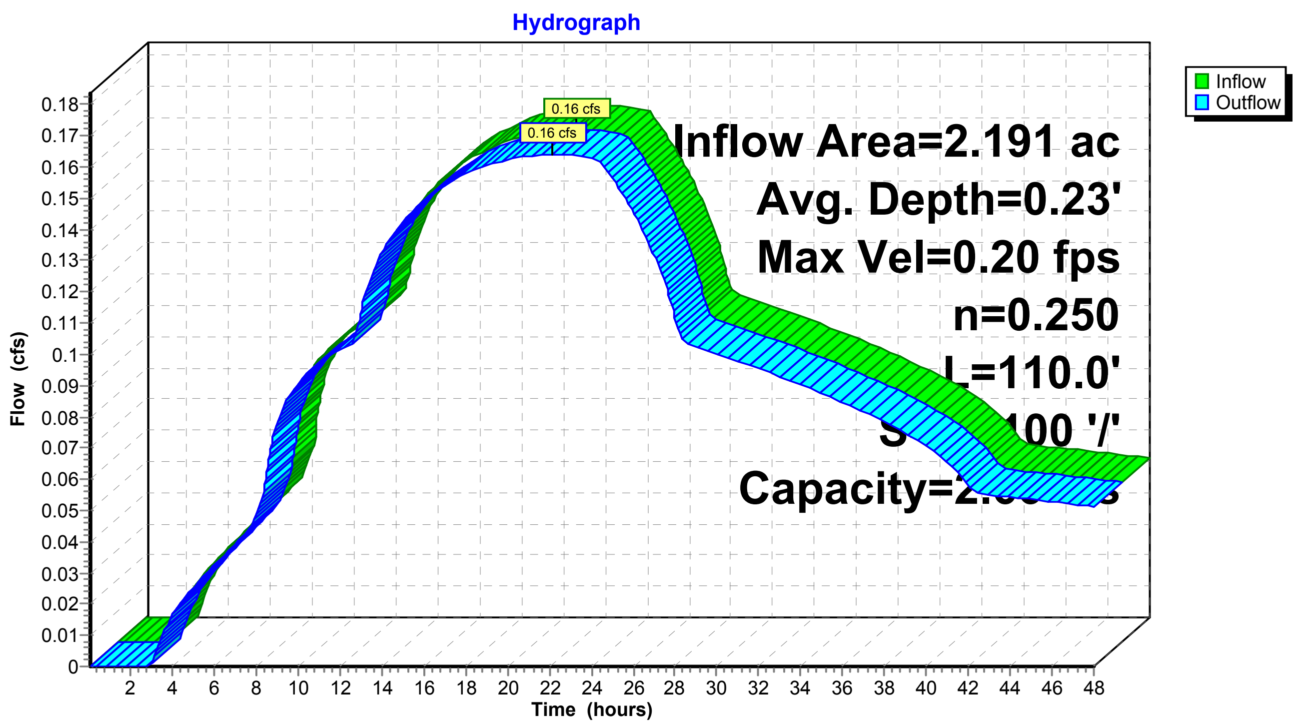
Routing by Stor-Ind+Trans method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs
Max. Velocity= 0.20 fps, Min. Travel Time= 9.4 min
Avg. Velocity = 0.16 fps, Avg. Travel Time= 11.6 min

Peak Storage= 92 cf @ 22.00 hrs, Average Depth at Peak Storage= 0.23'
Bank-Full Depth= 1.00', Capacity at Bank-Full= 2.66 cfs

3.00' x 1.00' deep channel, n= 0.250
Side Slope Z-value= 3.0 '/' Top Width= 9.00'
Length= 110.0' Slope= 0.0100 '/'
Inlet Invert= 426.00', Outlet Invert= 424.90'

‡

Reach 11R: Swale

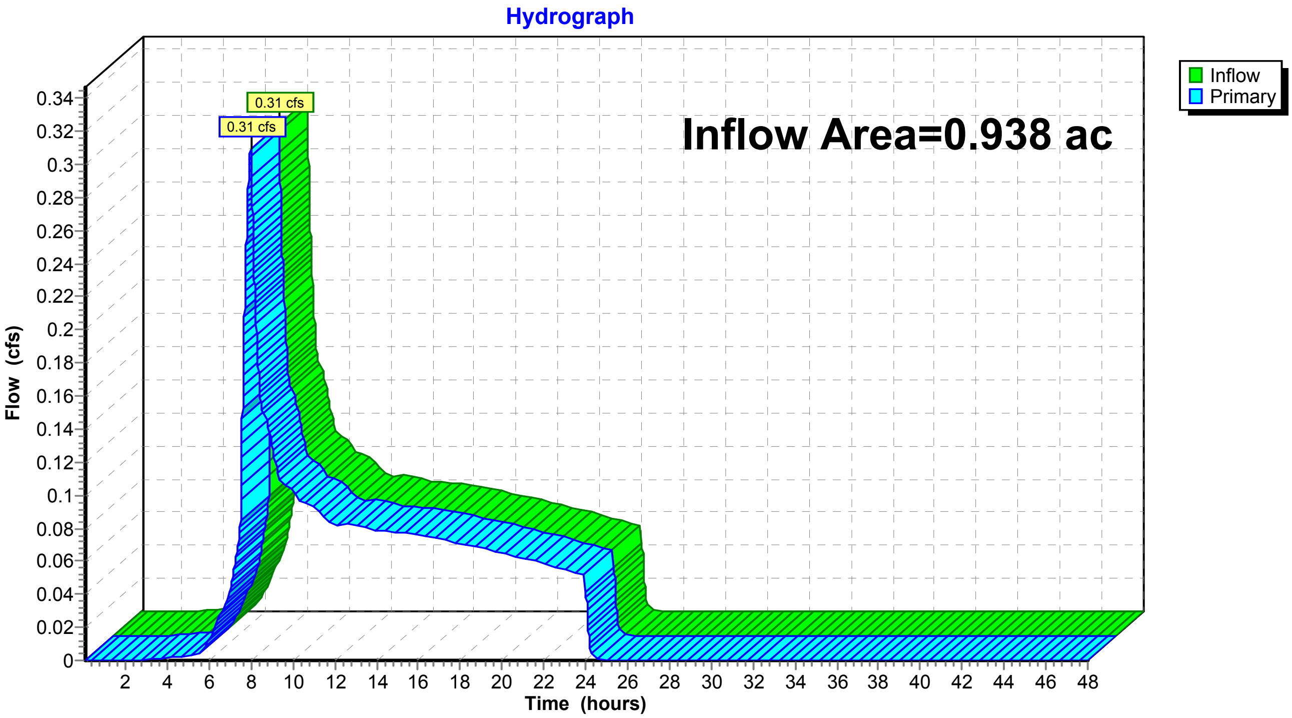


Summary for Pond 4P: Overland Runoff

Inflow Area = 0.938 ac, 0.00% Impervious, Inflow Depth = 1.58" for 25-year event
Inflow = 0.31 cfs @ 8.00 hrs, Volume= 0.123 af
Primary = 0.31 cfs @ 8.00 hrs, Volume= 0.123 af, Atten= 0%, Lag= 0.0 min

Routing by Stor-Ind method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs

Pond 4P: Overland Runoff



Summary for Pond 9P: Pond Outlet

Inflow Area = 2.191 ac, 35.37% Impervious, Inflow Depth = 2.50" for 25-year event
Inflow = 1.19 cfs @ 7.90 hrs, Volume= 0.456 af
Outflow = 0.16 cfs @ 21.90 hrs, Volume= 0.363 af, Atten= 86%, Lag= 839.9 min
Primary = 0.16 cfs @ 21.90 hrs, Volume= 0.363 af

Routing by Stor-Ind method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs
Peak Elev= 437.78' @ 21.90 hrs Surf.Area= 0.065 ac Storage= 0.263 af

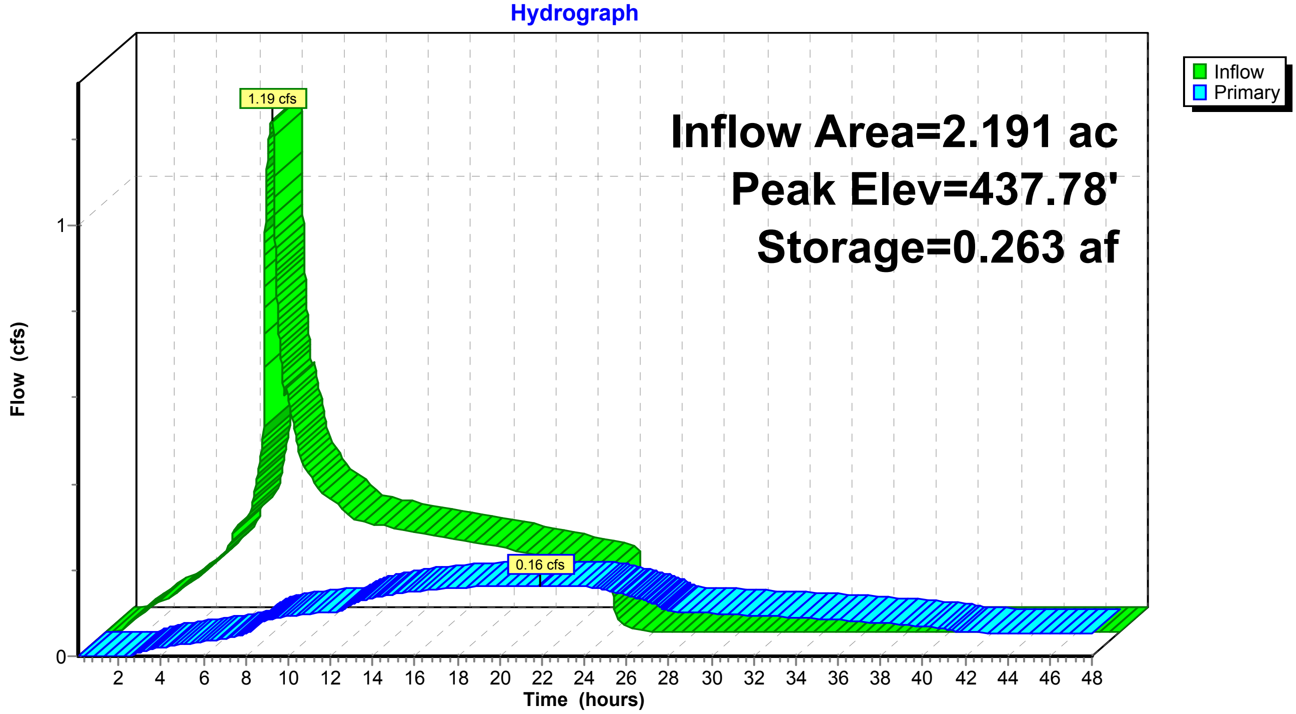
Plug-Flow detention time= 864.9 min calculated for 0.363 af (80% of inflow)
Center-of-Mass det. time= 734.4 min (1,468.4 - 734.0)

Volume	Invert	Avail.Storage	Storage Description
#1	430.00'	0.278 af	**15.00'W x 30.00'L x 8.00'H Prismatoid Z=2.0**

Device	Routing	Invert	Outlet Devices
#1	Primary	430.50'	**1.0" Vert. Orifice/Grate** C= 0.600
#2	Primary	435.00'	**1.0" Vert. Orifice/Grate** C= 0.600
#3	Primary	437.00'	**1.5" Vert. Orifice/Grate** C= 0.600
#4	Primary	438.00'	**45.0 deg x 3.0' long Sharp-Crested Vee/Trap Weir** C= 2.56

Primary OutFlow Max=0.16 cfs @ 21.90 hrs HW=437.78' (Free Discharge)
—**1=Orifice/Grate** (Orifice Controls 0.07 cfs @ 12.95 fps)
—**2=Orifice/Grate** (Orifice Controls 0.04 cfs @ 7.96 fps)
—**3=Orifice/Grate** (Orifice Controls 0.05 cfs @ 4.07 fps)
—**4=Sharp-Crested Vee/Trap Weir** (Controls 0.00 cfs)

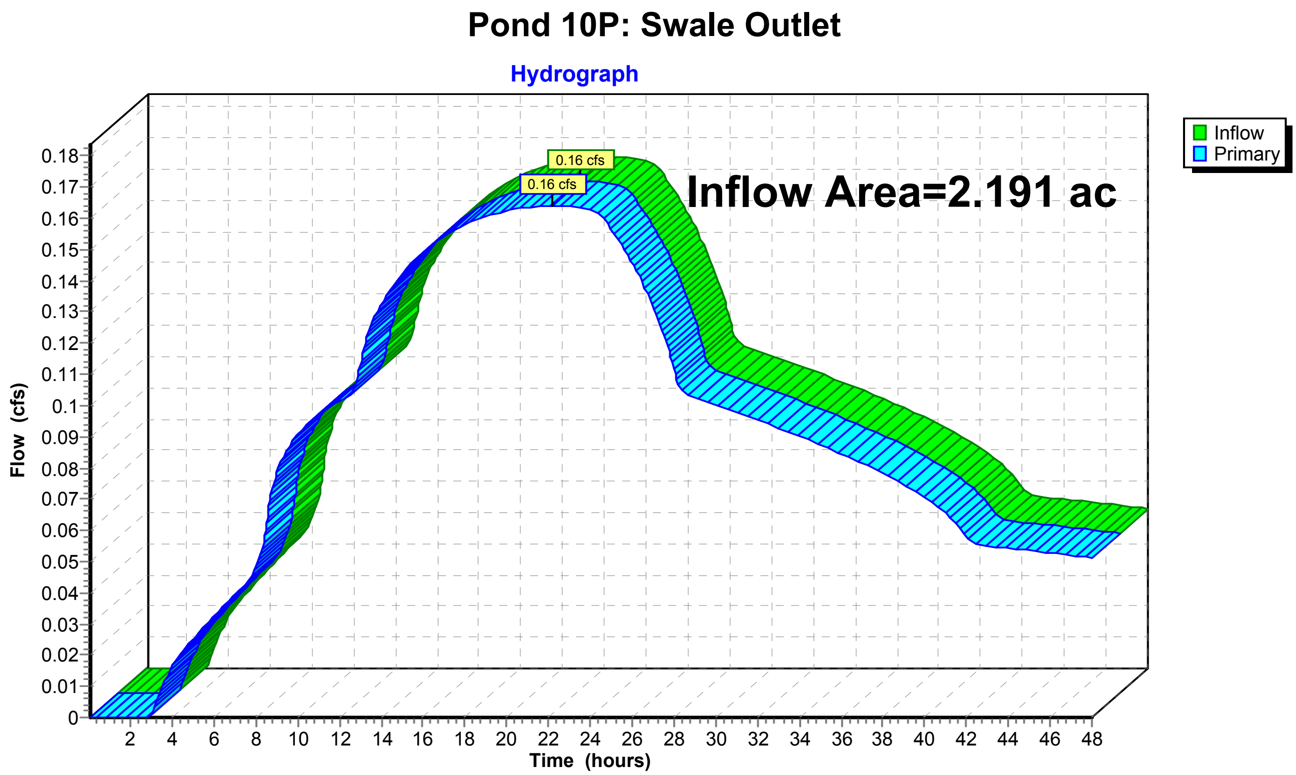
Pond 9P: Pond Outlet



Summary for Pond 10P: Swale Outlet

Inflow Area =	2.191 ac,	35.37% Impervious,	Inflow Depth > 1.98"	for 25-year event	
Inflow =	0.16 cfs @	22.16 hrs,	Volume=	0.362 af	
Primary =	0.16 cfs @	22.16 hrs,	Volume=	0.362 af,	Atten= 0%, Lag= 0.0 min

Routing by Stor-Ind method, Time Span= 0.08-48.00 hrs, dt= 0.01 hrs



Map Unit Description

The map units delineated on the detailed soil maps in a soil survey represent the soils or miscellaneous areas in the survey area. The map unit descriptions in this report, along with the maps, can be used to determine the composition and properties of a unit.

A map unit delineation on a soil map represents an area dominated by one or more major kinds of soil or miscellaneous areas. A map unit is identified and named according to the taxonomic classification of the dominant soils. Within a taxonomic class there are precisely defined limits for the properties of the soils. On the landscape, however, the soils are natural phenomena, and they have the characteristic variability of all natural phenomena. Thus, the range of some observed properties may extend beyond the limits defined for a taxonomic class. Areas of soils of a single taxonomic class rarely, if ever, can be mapped without including areas of other taxonomic classes. Consequently, every map unit is made up of the soils or miscellaneous areas for which it is named and some minor components that belong to taxonomic classes other than those of the major soils.

Most minor soils have properties similar to those of the dominant soil or soils in the map unit, and thus they do not affect use and management. These are called noncontrasting, or similar, components. They may or may not be mentioned in a particular map unit description. Other minor components, however, have properties and behavioral characteristics divergent enough to affect use or to require different management. These are called contrasting, or dissimilar, components. They generally are in small areas and could not be mapped separately because of the scale used. Some small areas of strongly contrasting soils or miscellaneous areas are identified by a special symbol on the maps. If included in the database for a given area, the contrasting minor components are identified in the map unit descriptions along with some characteristics of each. A few areas of minor components may not have been observed, and consequently they are not mentioned in the descriptions, especially where the pattern was so complex that it was impractical to make enough observations to identify all the soils and miscellaneous areas on the landscape.

The presence of minor components in a map unit in no way diminishes the usefulness or accuracy of the data. The objective of mapping is not to delineate pure taxonomic classes but rather to separate the landscape into landforms or landform segments that have similar use and management requirements. The delineation of such segments on the map provides sufficient information for the development of resource plans. If intensive use of small areas is planned, however, onsite investigation is needed to define and locate the soils and miscellaneous areas.

An identifying symbol precedes the map unit name in the map unit descriptions. Each description includes general facts about the unit and gives important soil properties and qualities.

Soils that have profiles that are almost alike make up a *soil series*. All the soils of a series have major horizons that are similar in composition, thickness, and arrangement. Soils of a given series can differ in texture of the surface layer, slope, stoniness, salinity, degree of erosion, and other characteristics that affect their use. On the basis of such differences, a soil series is divided into *soil phases*. Most of the areas shown on the detailed soil maps are phases of soil series. The name of a soil phase commonly indicates a feature that affects use or management. For example, Alpha silt loam, 0 to 2 percent slopes, is a phase of the Alpha series.

Some map units are made up of two or more major soils or miscellaneous areas. These map units are complexes, associations, or undifferentiated groups.

A *complex* consists of two or more soils or miscellaneous areas in such an intricate pattern or in such small areas that they cannot be shown separately on the maps. The pattern and proportion of the soils or miscellaneous areas are somewhat similar in all areas. Alpha-Beta complex, 0 to 6 percent slopes, is an example.

An *association* is made up of two or more geographically associated soils or miscellaneous areas that are shown as one unit on the maps. Because of present or anticipated uses of the map units in the survey area, it was not considered practical or necessary to map the soils or miscellaneous areas separately. The pattern and relative proportion of the soils or miscellaneous areas are somewhat similar. Alpha-Beta association, 0 to 2 percent slopes, is an example.

An *undifferentiated group* is made up of two or more soils or miscellaneous areas that could be mapped individually but are mapped as one unit because similar interpretations can be made for use and management. The pattern and proportion of the soils or miscellaneous areas in a mapped area are not uniform. An area can be made up of only one of the major soils or miscellaneous areas, or it can be made up of all of them. Alpha and Beta soils, 0 to 2 percent slopes, is an example.

Some surveys include *miscellaneous areas*. Such areas have little or no soil material and support little or no vegetation. Rock outcrop is an example.

Additional information about the map units described in this report is available in other soil reports, which give properties of the soils and the limitations, capabilities, and potentials for many uses. Also, the narratives that accompany the soil reports define some of the properties included in the map unit descriptions.

Clackamas County Area, Oregon

13B—Cascade silt loam, 3 to 8 percent slopes

Map Unit Setting

National map unit symbol: 2234
Elevation: 250 to 1,400 feet
Mean annual precipitation: 50 to 60 inches
Mean annual air temperature: 50 to 54 degrees F
Frost-free period: 165 to 210 days
Farmland classification: Prime farmland if drained

Map Unit Composition

Cascade and similar soils: 80 percent
Minor components: 3 percent

Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Cascade

Setting

Landform: Hillslopes
Landform position (two-dimensional): Footslope, summit
Landform position (three-dimensional): Interfluve, crest
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Silty material

Typical profile

H1 - 0 to 11 inches: silt loam
H2 - 11 to 21 inches: silt loam
H3 - 21 to 60 inches: silty clay loam

Properties and qualities

Slope: 3 to 8 percent
Depth to restrictive feature: 20 to 30 inches to fragipan
Natural drainage class: Somewhat poorly drained
Capacity of the most limiting layer to transmit water (Ksat): Moderately low to moderately high (0.06 to 0.20 in/hr)
Depth to water table: About 18 to 30 inches
Frequency of flooding: None
Frequency of ponding: None
Available water storage in profile: Low (about 4.0 inches)

Interpretive groups

Land capability classification (irrigated): 3w
Land capability classification (nonirrigated): 3w
Hydrologic Soil Group: C
Other vegetative classification: Poorly Drained (G002XY006OR)

Minor Components

Delena

Percent of map unit: 3 percent
Landform: Hillslopes, terraces
Landform position (two-dimensional): Footslope
Landform position (three-dimensional): Interfluve, riser
Down-slope shape: Linear
Across-slope shape: Linear
Other vegetative classification: Poorly Drained (G002XY006OR)

Data Source Information

Soil Survey Area: Clackamas County Area, Oregon
Survey Area Data: Version 9, Sep 19, 2014

SECTION 7
ENGINEERING CONCLUSIONS

The requirements for water quality are met by using the grassy swale standards to size the vegetative facilities. As outlined in the HydroCad modeling results, the flow control structures were sized so the post development runoff rates will not exceed the 5-, 10- and 25-year pre-developed runoff rates discharging to Bolton Creek. The piped post-development conveyance from the 2-year storm is half of the predevelopment rate.

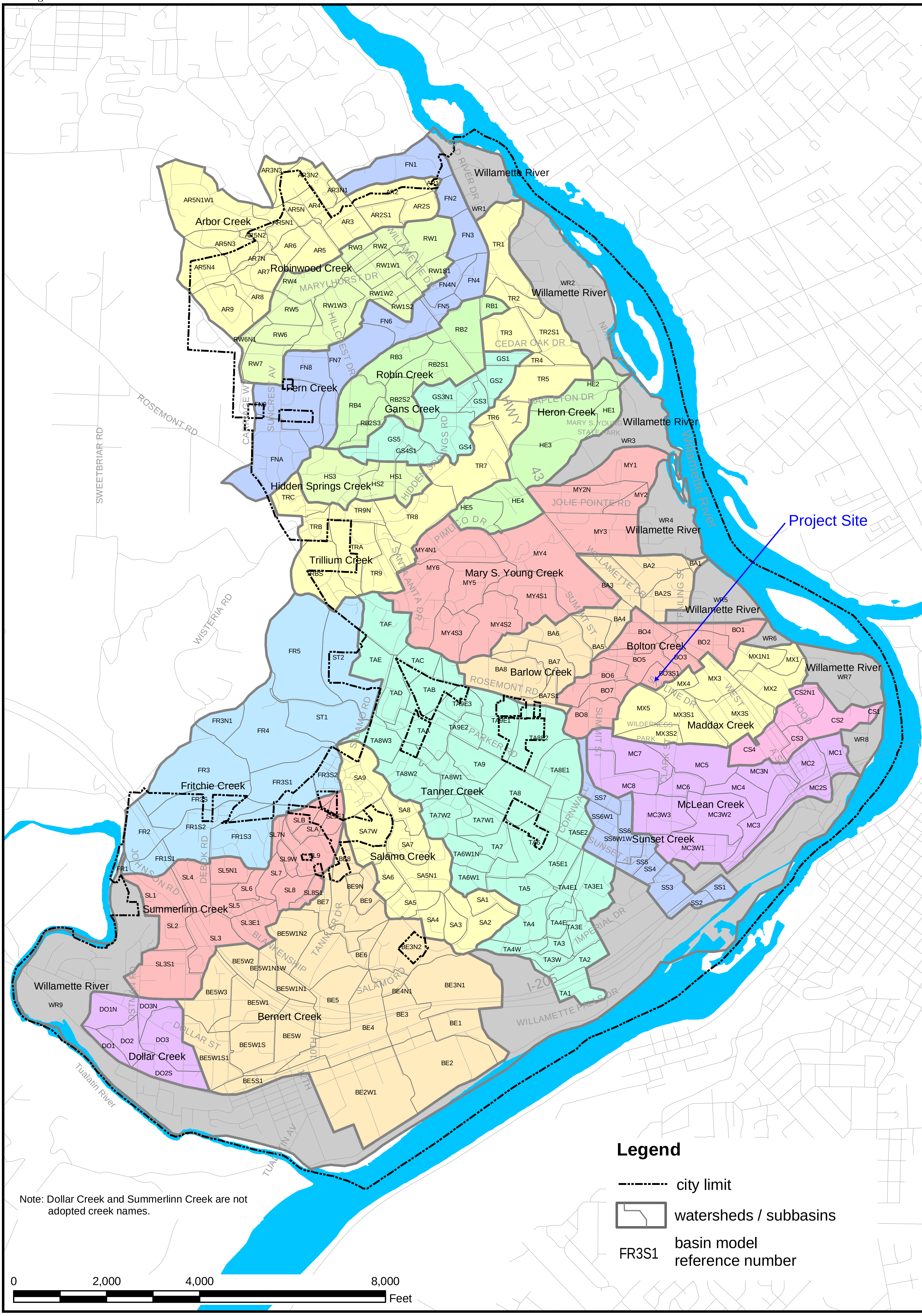
Overland post-development runoff is approximately half the pre-development rate for the 2-, 5-, 10- and 25-year storms. Total post-development peak runoff from the site is less than the pre-development condition.

SECTION 8
STORMWATER FACILITY DETAILS AND EXHIBITS

These exhibits to follow. Refer to the Table of Content for anticipated drawings and exhibits.

Surface Water Management Plan 2006

West Linn GIS
GEOGRAPHIC INFORMATION SYSTEMS



storm_mp2004\watershed_delineations.mxd | Lee | 9-14-06 (draft 8-03-04)

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Taxlot Base Source: Clackamas County GIS

Figure 4.1 Watershed Delineations

SECTION 9
OPERATIONS AND MAINTENANCE

<TO FOLLOW>

SECTION 10
ASSOCIATED REPORTS SUBMITTED

"Geotechnical Investigation and Site-Specific Seismic Hazard Study 4-MG Bolton Reservoir West Linn, Oregon", GRI, January 23, 2015.

Arborist Report, Morgan Holen & Associates, May 2015
Bolton Reservoir Replacement
West Linn, Oregon

GRI
9750 SW Nimbus Avenue
Beaverton, OR 97008-7172
p | 503-641-3478 f | 503-644-8034

January 23, 2015 5338-A GEOTECHNICAL RPT

Murray, Smith & Associates, Inc.
121 SW Salmon Street, Suite 900
Portland, OR 97204

DRAFT

Attention: Tom Boland, PE

**SUBJECT: Geotechnical Investigation and Site-Specific Seismic Hazard Study
4-MG Bolton Reservoir
West Linn, Oregon**

At your request, GRI has conducted a geotechnical investigation and site-specific seismic hazard study for the above-referenced project in West Linn, Oregon. The general location of the site is shown on the Vicinity Map, Figure 1. The purpose of this investigation was to evaluate subsurface materials and conditions at the site and develop geotechnical recommendations for use in design and construction of the reservoir. The investigation included a review of available geotechnical information for the site and vicinity, subsurface explorations, laboratory testing, and engineering and seismic analyses. This report describes the work accomplished and provides our conclusions and recommendations for design and construction of the proposed reservoir.

Because the reservoir is considered an essential facility in accordance with the 2014 Oregon Structural Specialty Code (OSSC), our investigation included a site-specific seismic hazard study.

GRI completed a preliminary geotechnical evaluation of the site to support the conceptual siting analysis. The results of our evaluation are summarized in our August 31, 2012, report to Murray, Smith & Associates, Inc. entitled, "Preliminary Geotechnical Evaluation for Conceptual Siting Analysis, 4-MG Bolton Reservoir, West Linn, Oregon."

PROJECT DESCRIPTION

As currently proposed, the existing 2.5-million gallon (MG) concrete reservoir will be replaced with a partially embedded 4-MG concrete reservoir established in a cut up to 30 ft deep. The approximate location of the proposed tank with respect to the existing reservoir and site topography is shown on the Site Plan, Figure 2. The new reservoir will consist of a partially embedded, American Water Works Association (AWWA) D110-13 Type I wire-wound, circular, pre-stressed concrete tank with an inside diameter of about 168 ft. The January 12, 2015, pre-design report by Peterson Structural Engineers (PSE) indicates the floor of the new reservoir will be established at approximate elevation 425 ft (NAVD 88) and overflow at approximate elevation 451 ft with 2 ft of freeboard. The reservoir foundation was originally designed to be a 24-in.-thick reinforced concrete mat slab. The 9-in.-thick reinforced concrete roof will be supported by a 12-in.-thick core wall and 24-in. diameter columns on approximately 20-ft center-to-center spacing. The tank will be backfilled to elevation 442 and 445 ft on the north and south side, respectively, and will support a 15-ft-wide gravel service road.

As shown on Figure 2, the new reservoir will be established toward the southwest portion of the site, and the northern side of the reservoir will be about 50 ft farther south than the existing reservoir to reduce the risk of potential local slope instability along the north side of the site. The top of the slope along the north side of the site will be flattened by removing soil to improve the overall stability of the slope.

Based on our experience with similar projects, the amount of differential settlement that can be tolerated across the footprint of a concrete reservoir is small, and limiting differential settlement will be critical to the performance of the reservoir. Possibly poor quality fill and localized zones of soft, compressible soil in the basalt have been disclosed by recent exploration. To reduce the risk of undesirable settlement beneath the reservoir, ground improvement, such as rammed aggregate piers overlain with several feet of compacted crushed rock, is planned to limit settlement to acceptable levels. Ground improvement is also required to improve the factor of safety for the seismic slope stability.

The excavation necessary for construction of the new reservoir is anticipated to extend to approximately 30 ft below existing grades. As currently planned, the side slopes of the excavation will be sloped at up to 1H:1V where space allows. However, we anticipate a shoring system constructed from the top-down, such as a tied-back soldier pile wall or possibly a soil-nail wall, may be necessary to retain the temporary excavation next to the existing pump station to the southeast and along the west side of the reservoir footprint near the properly line. We anticipate the shoring walls could have a total retained height of up to 30 ft.

The project will also include construction of new piping, a valve vault, landscaping, and a gravel access road. The 18-in.-diameter inlet/outlet line in Skyline Circle will be replaced with a 24-in.-diameter line, and the existing 8-in.-diameter PVC main north of the reservoir will be replaced with an 8-in.-diameter ductile iron line. A new overflow line will be constructed at a location that has not yet been determined.

SITE DESCRIPTION

Topography and Surface Conditions

As shown on the Site Plan, Figure 2, and the Site Map, Figure 3, the reservoir site is located northeast of Skyline Drive on a relatively flat bench at about elevation 445 to 450 ft (NAVD 88). Land use in the area surrounding the existing reservoir consists of forested undeveloped land to the south and residential to the west, north, and east. The ground surface north of the reservoir slopes downward at about 25° to the northeast to residences along Caufield Street and is vegetated with mature trees and brush.

GEOLOGY

Geologic Setting

The site is located on the eastern flank of the Tualatin Mountains, a topographic upland that separates the Portland Basin to the northeast from the Tualatin Basin to the west and the Willamette Valley to the south. Geologic mapping completed for the area indicates the site is located in the vicinity of the contact between the Miocene Wanapum Basalt and the Grande Ronde Basalt units of the Columbia River Basalt Group (Madin, 2009). Where fresh and unweathered, these basalt units are typically a light to dark gray, dense volcanic rock. However, the Wanapum-Grande Ronde boundary is characterized in places by an erosional unconformity or an interbed that varies from non-marine sediments to a thick relic soil, and is referred to as the Vantage Horizon (Beeson et al., 1985). The Vantage Horizon originated during a period of erosion and soil development that occurred between volcanic flow events. Large-scale landslides are known to occur where the Vantage Horizon daylights at or near the ground surface. The reservoir site and other areas of the Tualatin Mountain upland are capped by deposits of fine-grained, windblown silt, referred to as Portland Hills Silt. Quaternary alluvial deposits associated with the Willamette River and the Ice Age Missoula Floods (about 15,000 to 20,000 years ago) are present northeast of the site, north of Highway 43. A geologic map and cross section of the project area are provided on Figure 4.

Faults

General. Several geologic faults are located in the project area. Two northeast-trending unnamed normal faults are mapped near the site (Yeats et al., 1991). These faults, which are bedrock faults in the Columbia River Basalt, do not have historic seismicity and are not considered by U.S. Geological Survey (USGS) to contribute to the seismic hazard at the site. The surface trace of the Bolton Fault is located about 900 ft northeast of the site, the Oatfield Fault is about 2.5 miles northeast of the site, and the Portland Hills Fault is about 3 miles northeast of the site (Schlicker and Finlayson, 1979; Personius et al., 2002). These faults do not have historic seismicity, but the USGS considers each of these faults to contribute to the overall seismic hazard at the site.

Bolton Fault. The northwest-trending Bolton Fault is responsible for the straight, abrupt front of the hills west of Highway 43 between Lake Oswego and West Linn. The Bolton Fault does not appear to have moved since the time of the Missoula Floods, about 15,000 to 20,000 thousand years ago (DOGAMI, 2009). This fault is located about 900 ft northeast of the site. USGS considers the structure a southwest-dipping reverse fault with down-to-the-northeast separation of up to 200 m (600 ft) in Miocene volcanic rocks (Personius et al., 2002). No fault scarps in surficial deposits or other unequivocal evidence of Quaternary displacement has been described in the literature. The USGS classifies the fault as Class B until further studies are conducted (Personius et al., 2002). Class A faults generally have a slip rate greater than 5 mm/yr and well constrained paleoseismic data. Class B faults include all other faults lacking paleoseismic data necessary to constrain the recurrence intervals of large events (Petersen et al., 1996).

An online Department of Geology and Mineral Industries (DOGAMI) mapping viewer (DOGAMI HazVue, accessed January 8, 2015) places the closest point of the surface trace of the Bolton Fault about 900 ft northeast of the existing reservoir (distance measured from northeast corner of existing reservoir to the trace mapped at intersection of Highway 43 and Buck Street). Other published DOGAMI maps show the surface trace of the Bolton Fault generally coincident with the relatively linear eastern slope toe of the Tualatin Mountains upland, or about 900 ft northeast of the existing reservoir (Schlicker and Finlayson, 1979, scale 1:24,000; Burns et al., 1997, scale 1:100,000). However, it should be noted that the available geologic resolution and confidence to locate the Bolton Fault with about 500 ft at scales of 1:24,000 and 1:100,000 is low. Yeats et al. (1991) and Madin (2009) map two strands of the Bolton Fault near the site, see Figure 4. Their mapping shows one strand along the abrupt topographic escarpment, and another buried strand is concealed beneath Quaternary alluvial deposits near Highway 43.

Canby 133 Ancient Landslide

DOGAMI is the state agency responsible for geologic hazard mapping in Oregon. DOGAMI has indicated in its statewide landslide hazard database that Bolton Reservoir is located on a prehistoric (>150 yrs), deep-seated (>15 ft deep), translational rock landslide, referred to as Canby 133. Figure 5 shows the limits of the landslide from the state database. Mapping of landslide deposits are based, in part, on light detection and ranging (lidar) derived elevation data and interpretation of surface topography typical of landslide features. Canby 133 was mapped using lidar and a method protocol outlined by DOGAMI (2009) with a "moderate" level of confidence. The confidence ranking (low, moderate, and high) is based on desktop analysis. Bill Burns with DOGAMI was contacted regarding this feature and recalls they did a vehicle-based reconnaissance from public roads to map this feature, but he was not aware of other data (i.e., reports, borings, or anecdotal stories of ground movement) about the feature. Mr. Burns indicated unpublished DOGAMI field mapping from 2004 also indicates the area is a landslide. This information suggests the Bolton Reservoir site is located on a very large, old or "ancient" landslide.

As part of the Murray, Smith & Associates, Inc. (MSA) team, Cornforth Consultants, Inc. (2014) completed a seismic landslide evaluation for the planned reservoir. The evaluation was performed to identify any signs of landslide activity near the reservoir and to provide opinions on potential impacts of seismic landslide displacements on proposed improvements at the site. Their geotechnical reconnaissance of the ancient landslide around Bolton Reservoir did not identify signs of active movement, especially along the margins, where differential movement would be greatest. They also concluded the ancient landslide is likely to move feet rather than tens of feet during a large earthquake.

The mapped northeast boundary of the Canby 133 landslide near the site is essentially coincident with the prominent straight and abrupt topographic escarpment associated with the Bolton Fault. In our opinion, this indicates the Bolton Fault cross-cuts the toe of the Canby 133 landslide. Therefore, the Canby 133 landslide is likely on the order of at least 15,000 to 20,000 years old.

SLOPE STABILITY

Previous Reports

Three geotechnical engineering reports prepared for the Bolton Reservoir site in 1972, 1988, and 1998, were provided to GRI. The first report was prepared by Northwest Testing Laboratories (NTL) for the City of West Linn (City) in 1972 (NTL, 1972). The report provided the results of a soil and foundation investigation and recommendations for enlarging the reservoir. The report concluded the slope east of the site could accommodate the additional load of the reservoir.

L.R. Squier Associates, Inc. prepared a geologic reconnaissance report for the City in 1988 (L.R. Squier, 1988). The purpose of the report was to evaluate the slope northeast of the reservoir for a planned residential development, where there were concerns of slope stability. The report concluded that steep slopes, weak and locally thin soils, soil creep, and groundwater seepage from springs suggested a high risk for slope instability, and a comprehensive geotechnical investigation was recommended.

In the 1970s, a small earth flow landslide occurred along the steeply sloping wooded area northeast of the reservoir. Large ground cracks occurred north of the reservoir in 1996 following heavy rainfall. Landslide Technology conducted an investigation into the stability of the steep slope area in 1997 (Landslide Technology, 1998). The investigation included a reconnaissance, subsurface explorations, laboratory testing, installation of an open-pipe piezometer and inclinometer casing. Based on the results of the investigation, the report provided an approach for repair of the small earth flow failure.

Site Reconnaissance

A reconnaissance of the site and surrounding area was conducted by a registered geologist and a certified engineering geologist from GRI in June 2012 and January 2015. The following description of the site is a summary of the observations made during the reconnaissance activities. Private properties located immediately northwest and southeast of the site were not accessed, but observed from the public right of way for features of significance. To the northeast, the ground slopes downward at approximately 25° toward Caufield Street. The slope northeast of the reservoir site is wooded with predominantly deciduous trees and occasional conifer tree, and springs. The ground surface is generally covered by English ivy, ferns, and blackberries. Several springs and flowing water were also observed along Caufield Street and originated from the slope above. A concrete manhole and pipe valve were observed along the slope near the northern property boundary. The valve appeared to be rusted through and was leaking water. No indications of recent slope instability were observed along the northeast slope during the site reconnaissance. The surrounding neighborhood was also examined from the public right of way for indications of slope movement (cracked and separated sidewalks or curbs). The reconnaissance did not disclose obvious indications of relatively recent movement, such as cracked streets, sidewalks, or curbs. Limited interviews with City maintenance personnel did not disclose reports of broken or sheared underground utilities.

The slope failure that occurred along the northeast side of the existing reservoir in 1996 and investigated by Landslide Technologies has not been repaired and is covered with vegetation as observed during our January 2015 reconnaissance. Most of the remainder of the slope along the north side of the reservoir has the same general appearance and inclination of the slope adjacent to the landslide. The existing reservoir was fully covered with a liner and could not be examined. However, cracking is present along portions of the north side of the reservoir flatwork and ring wall, particularly in the northwest corner. As with previous observations in 2012, whether the flatwork and ring wall cracking is due to slope movement or fill settlement could not be ascertained.

Inclinometer

In June 2012 and November 2014, GRI monitored the inclinometer that was installed by Landslide Technologies in 1997 at the approximate location shown on Figure 2 during their evaluation of local instability at the northeast corner of the site. An inclinometer casing consists of a plastic pipe with a pair of orthogonal slots, or grooves, that permit a calibrated instrument to be lowered to the bottom of the casing. When the ground surrounding the casing moves, the casing distorts above the zone of movement, and the orientation of the casing changes. The orientation of the casing is measured by lowering the calibrated instrument to the bottom of the casing and reading the instrument at 2-ft intervals as it is withdrawn. The zone and rate of movement can be determined by comparing the results of successive sets of readings. The inclinometer was installed east of the proposed tank footprint to provide long-term monitoring of the site with respect to potential slope movement.

GRI obtained the baseline measurements collected by Landslide Technologies in 1997 and compared those data with measurements obtained from the inclinometer in June 2012 and November 2014. The readings indicate very small creep-type slope movements have occurred since the inclinometer casing was installed in 1997. The measurements indicate cumulative horizontal movement of 1 and 1.25 in. at the ground surface between the 1997 base line reading and the readings by GRI in June 2012 and November

2014, respectively. The majority of the movement occurred in the upper approximately 10 to 12 ft of the soil profile and was less than about 0.25 in. below this depth. The movement detected in the inclinometer gradually decreases with depth, to no obvious movement at a depth of about 40 ft. Indications of obvious movement at the ground surface, such as ground cracks or settlement, have not been observed during our recent visits to the site.

In our opinion, information provided in the report by Landslide Technology and monitoring of the inclinometer indicate the slope instability that occurred in 1996 is likely related in part to the presence of fill soil placed along the northern slope during the original construction of the reservoir. As part of the reservoir replacement project, soil will be removed from the top of the slope to improve local stability, which may impact the existing inclinometer and piezometer installed by Landslide Technology. We recommend preserving the slope inclinometer and piezometer for future monitoring. In this regard, the upper portion of the inclinometer and piezometer may need to be removed followed by a new inclinometer base line reading. GRI should participate closely with any field modifications to the inclinometer and piezometer casing.

SUBSURFACE CONDITIONS

General

Subsurface materials and conditions at the site were evaluated by GRI on June 15, 2012, with one boring, designated B-1, and on October 27 through 29, 2014, with two borings, designated B-2 and B-3. The locations of the borings are shown on Figure 2. The borings were advanced to depths rof about 76 to 90 ft. The field and laboratory programs completed for this study are discussed in detail in Appendix A. Logs of the borings are provided on Figures 1A through 3A. The terms and symbols used to describe the soil and rock encountered in the borings are defined in Tables 1A and 2A and the attached legend.

In addition to the borings completed by GRI, Landslide Technology (1998) and Northwest Testing Laboratories (1972) completed borings at the locations shown on Figure 2. Logs of the previously drilled borings are provided in Appendix B.

The explorations indicate the reservoir site is mantled with a variable thickness of silty and clayey manmade fill, underlain by native silty and clayey soils, which are in turn underlain by basalt of the Columbia River Basalt Group. The relative consistency of the silty and clayey fill and native soil is generally medium stiff to stiff. The native soil is underlain by extremely soft (R0), predominantly decomposed to decomposed basalt (Wanapum Basalt). The basalt has generally weathered to the consistency of medium stiff to hard soil. Localized zones in the decomposed basalt have weathered to the consistency of soft, silty and clayey soil. The soft soil-like zones were encountered locally between depths of about 20 and 40 ft below the ground surface. The basalt transitions to generally fresh to moderately weathered, medium hard to hard (R3 to R4) basalt at depths of about 55 to 60 ft below the ground surface. The Wanapum Basalt transitions to the Vantage Horizon of the Grande Ronde Basalt at a depth of about 79 and 71 ft below the ground surface in GRI borings B-2 and B-3, respectively. The zone between the two basalt formations is called the Vantage Horizon and consists of moderately weathered, very soft to medium hard (R1 to R3) basalt. GRI borings B-2 and B-3 did not disclose indications of soft soil and/or shear zones within the Vantage Horizon. The transition from soil-like weathered basalt to relatively intact medium hard to hard basalt at a depth of about 55 to 60 ft is interpreted to be the lower boundary of material within the mass of the very large, presently inactive, ancient/prehistoric, deep-seated landslide.

Groundwater

An observation standpipe piezometer was installed in GRI borings B-2 and B-3 to a depth of 90 and 48 ft, respectively, to monitor groundwater levels at the site. As discussed previously, Landslide Technology installed a standpipe piezometer to a depth of 40 ft in a boring at the northeast corner of the site. On November 18, 2014, groundwater levels in standpipe piezometers installed GRI borings B-2 and B-3, and Landslide Technology boring LT-1P were measured at depths of about 23, 42, and 19 ft, respectively, below the ground surface. On January 7, 2015, the groundwater level in borings B-2, B-3 and LT-1P was about 23, 41, and 19 ft, respectively, below the ground surface. We anticipate the regional groundwater level is significantly deeper, and the groundwater levels measured in the standpipes are perched within the soil and rock. It is expected that perched groundwater in the soil could approach the ground surface locally during periods of prolonged or intense precipitation that are common during the wet, fall through spring months and will likely drop to depths greater than 20 ft during typical dry, summer and early fall months.

CONCLUSIONS AND RECOMMENDATIONS

General

The new reservoir will be constructed toward the southwest portion of the site in a cut up to 30 ft deep and will have a finished floor at about elevation 425 ft and an overflow at elevation 451 ft with 2 ft of freeboard. The sides of the new reservoir will be backfilled to within about 5 to 10 ft of the top of the reservoir. To provide satisfactory seismic slope stability for the new reservoir and limit differential static settlements, ground improvement will be completed beneath the new tank, and soil will be removed along the crest of the slope along the north side of the site. Drainage will be installed around and beneath the reservoir to manage subsurface water, and new inlet/outlet and overflow piping will be installed.

The reservoir site is mantled with a variable thickness of relatively stiff, silty and clayey manmade fill that is underlain by relatively stiff, native silty and clayey soils, which are in turn underlain by basalt. The basalt has generally weathered to the consistency of medium stiff to hard soil to depths of about 55 to 60 ft. However, localized zones in the decomposed basalt between depths of about 20 to 40 ft have weathered to the consistency of soft, silty and clayey soil. Soft to hard (R2 to R4) basalt underlies the decomposed basalt at depths of 55 to 60 ft. The groundwater level at the site may approach the ground surface during periods of prolonged or intense precipitation that are common during the wet, fall through spring months.

As previously discussed, the reservoir site is located on a very large, ancient landslide. However, reconnaissances by GRI as part of this study and during our 2012 study did not disclose indications of recent landslide movement. A reconnaissance recently completed by Cornforth Consultants (December 2014) also did not identify signs of active movement. It is our opinion the risk of significant future movement of the large, ancient landslide is low. It is expected that the greatest risk of significant movement of the large landslide would be during and/or following a large seismic event. Because the reservoir site is located within the middle of this large translational landslide mass and away from the margins, the risk of significant differential movement within the footprint of the new reservoir following the design-level earthquake is expected to be low. The planned ground improvement beneath the reservoir, removal of soil at the top of the slope along the north side of the site, and the gravel pad and subdrainage system around and beneath the reservoir will improve local factors of safety as they relate to potential reservoir instability. In our opinion, the new reservoir, as planned, will not adversely affect the existing site slope stability. Slope stability analyses and discussion are provided in the Slope Stability Analyses section in this report.

In our opinion, the proposed reservoir can be supported on spread footings and a reinforced floor slab system underlain by a granular base course section underlain by improved ground. We anticipate overall site grading can be accomplished with conventional construction equipment. The major geotechnical considerations with construction of the planned reservoir are the moisture-sensitive nature of the soil and decomposed basalt and potential for shallow, perched groundwater. The following sections of this report provide our conclusions and recommendations for design and construction of the reservoir.

Seismic Considerations

We anticipate the new reservoir will be designed in accordance with the AWWA D110-13 standard entitled, *Wire- and Strand-Wound, Circular, Prestressed Concrete Water Tanks*, and the 2012 International Building Code (IBC) with 2014 Oregon Structural Specialty Code (OSSC) modifications. The 2012 IBC evaluates seismic loading in accordance with the American Society of Civil Engineers (ASCE) 7-10 document entitled, *Minimum Design Loads for Buildings and Other Structure*.. We anticipate seismic design of the new reservoir will be completed in accordance with the 2012 IBC and ASCE 7-10 documents.

The reservoir is considered an essential facility by Oregon Revised Statute (ORS) 455.447, and GRI has completed a site-specific seismic hazard study in accordance with the 2012 IBC with 2014 OSSC modifications. The results of this study are provided in Appendix B and indicate IBC Site Class D, or a stiff soil site, is appropriate for design of the new reservoir. The IBC design methodology uses two spectral response coefficients, S_S and S_1, corresponding to periods of 0.2 and 1.0 second, to develop the MCE_R earthquake spectrum. The S_S and S_1 coefficients for the site located at the approximate latitude/longitude coordinates of 45.37° N and 122.63° W are 0.95 and 0.41 g, respectively. We recommend using the code-based F_a and F_v factors of 1.12 and 1.59, respectively, for Site Class D conditions to estimate the ground surface response spectrum. The design spectrum is based on a damping ratio of 5%. To evaluate sloshing at a damping ratio of 0.5%, the design spectrum for Site Class D can be multiplied by a factor of 1.5.

Based on preliminary evaluations, there is some risk of seismically induced soil strength loss in relatively thin zones in the decomposed basalt that have weathered to the consistency of soft soil that were encountered locally between depths of about 25 to 40 ft below the existing ground surface. In our opinion, the risk of significant post-earthquake settlement due to soil strength loss in these isolated layers is low. However, the presence of these layers presents a risk of seismic slope instability. A discussion of slope stability and alternatives to reduce the risk of instability are provided below.

The risk of damage by tsunami and/or seiche at the site is absent due to the elevation of the site. In our opinion, the risk of liquefaction-induced lateral spreading and ground deformation at the site is very low. As previously discussed, the surface trace of the Bolton Fault is about 900 ft northeast of the site. Unless occurring on a previously unmapped or unknown fault, it is our opinion the risk of ground rupture at the site is low. In our opinion, there is a risk of seismically induced localized slope instability at the site; however, we anticipate the proposed ground improvement program discussed in the following sections will be completed to reduce the risk of seismic slope instability to an acceptable level. Additional discussion of local faults and other seismic considerations is provided in Appendix C.

Slope Stability Analyses

As discussed previously, the silty and clayey soil that mantles the site is relatively stiff, and the underlying decomposed basalt typically has a consistency comparable medium stiff to hard soil. However, localized zones in the decomposed basalt have weathered to soft, silty and clayey soil between depths of about 20 to 40 ft below the ground surface. It is possible that these soft zones in the decomposed basalt could extend laterally beneath the site and present a potential risk for localized slope instability, particularly during the design-level earthquake.

Slope stability analyses were completed to evaluate the potential risk of local slope instability affecting the new reservoir. The location of the assumed critical cross section used to develop the slope stability models is shown on Figure 2 and is oriented in a general south-north direction through the center of the planned reservoir, where the side of the reservoir is closest to the slope along the north side of the site. Models were developed to evaluate slope stability for the proposed reservoir (without and with ground improvement) and the existing reservoir. The stability models developed are shown on the Slope Stability Models, Figures 6 through 9. The slope stability models were analyzed with the aid of the computer software SLOPE/W by GeoSlope International of Calgary, Alberta, Canada. The groundwater level and locations/boundaries of soil and rock units and associated physical properties used in the models are provided on the aforementioned figures. The new reservoir was assumed to have a reinforced-concrete bottom thickness of 24 in. underlain by a 3-ft-thick crushed rock base course/drainage section. A horizontal pseudo-static coefficient of 0.22 (k_h) for the design-level earthquake, which is equal to about half of the design-level PGA (required by the 2014 OSSC), was used to evaluate the seismic factor of safety values. A residual internal angle of friction of 21° and 0 psi cohesion were used to model potential soft zones that may be present in the decomposed basalt layer, based on torsional ring shear residual strength testing of a sample of soft, clayey silt obtained from within the decomposed basalt at a depth of about 35 ft in boring B-2. The results of this testing are provided in Appendix A.

For the configurations and assumptions described above, and as shown on Figures 6 through 9, a factor of safety against local slope instability for seismic conditions was first computed for potential failure surfaces that could extend laterally beneath the new and existing reservoir. The computed factor of safety against instability is defined as the ratio of the forces (or moments) tending to resist failure to the forces (or moments) tending to cause failure. Computed factors of safety less than 1.0 represent potentially unstable conditions. Based on site geometry and subsurface conditions, it is assumed the most likely mode of failure will consist of translational block-type failures. As shown on Figure 6, the results of the modeling indicate a local seismic factor of safety of 1.0 for a potential slip surface that extends through potential soft zones in the silt and decomposed basalt beneath the new reservoir. A minimum factor of safety of 1.1 against seismic slope instability is typically used for design. To improve the local seismic factor of safety, ground improvement was assumed to be completed beneath the reservoir extending to an average depth of about 20 ft below the base of the reservoir and through potential soft zones observed in the borings to the top of the harder decomposed basalt. For the purpose of analysis, it is assumed the ground improvement will likely consists of rammed aggregate piers (Geopiers or similar) with a 30% replacement ratio. The replacement ratio is the area of improved ground (aggregate piers) relative to the total area. It is further assumed the aggregate piers will have an effective stress internal angle of friction of at least 45°, resulting in the improved zone having an equivalent average effective stress internal angle of friction of 29°. As shown on Figures 7 and 8, the ground improvement zone in the model was assumed to extend 10 and 20 ft horizontally beyond the south and north side of the reservoir, respectively. As shown on Figure 7, a minimum seismic factor of safety of 1.1 against instability was computed for slip surfaces extending from south to north under the reservoir, assuming completion of ground improvement. As shown on Figure 8, the seismic factors of safety for potential slip surfaces on the sloping ground along the north side of the site that could potentially extend under the reservoir are greater than 1.5, assuming ground improvement is completed. For comparison purposes, a slope stability model for the existing reservoir was also developed and is shown on Figure 9. The minimum seismic factor of safety against instability computed for a potential slip surface extending south to north under the existing reservoir is about 0.7 and is notably lower than for the planned reservoir constructed either without or with ground improvement. The primary reasons the new reservoir has a greater factor of safety than the existing reservoir, even without ground improvement, are the new reservoir will be set back a greater distance from the slope along the north side of the site, the drainage layers beneath and around the new reservoir will maintain a lower local groundwater level, and there will be an overall net decrease in gravity loads since the new reservoir will replace a significant amount of heavier excavated soil.

The results of our stability analysis indicate ground improvement will be necessary beneath the new reservoir to achieve a satisfactory seismic factor of safety against local instability that could affect the new reservoir. A discussion of recommended ground improvement is provided in the next section. Additionally, the top of the slope along the north side of the site should be flattened as much as practical by removal of soil. The planned flattening of the top of the slope along the north side of the site will lower the soil loads and improve the overall stability of the sloping ground north of the reservoir and, consequently, will reduce the risk of relatively shallow failures like those that occurred at the northeast corner of the site in the 1970s and in 1996. We recommend the subsurface drains under and around the reservoir, and surface drainage, be collected and discharged to an appropriate off-site location.

In our opinion, the measures discussed above will provide a satisfactory factor of safety against local instability affecting the new reservoir, but will not mitigate potential movements of the ancient large slide mass. Due to the large size of the landslide and potential deep failure surfaces, mitigation measures to improve the stability of the large landslide mass are likely not practical or cost effective. As discussed previously, obvious indications of recent movement of the large landslide mass were not observed during site reconnaissances completed by GRI and Cornforth Consultants, nor have there been reports of potential movements of the large landslide. Based on the available information, the risk of significant movement of the large landslide within the design life of the reservoir is expected to be low and would most likely occur during/following a large seismic event. It is expected that if movement of the large landslide mass occurs, the ground supporting the reservoir will tend to "raft" along with the greater landslide mass and the risk of significant differential movements beneath the reservoir will be reduced. In addition, the proposed ground improvement will strengthen the ground beneath the reservoir, which will further reduce the risk of significant differential movements.

Ground Improvement

As discussed in the previous section, ground improvement will be required beneath the new reservoir to improve seismic slope stability and limit static differential settlement. We anticipate the ground improvement will need to extend to depths of about 20 to 25 ft beneath the base of the new reservoir and through potential soft zones in the decomposed basalt to the top of harder basalt. Based on the subsurface conditions, site constraints, and cost, we anticipate rammed aggregate piers (RAP) or similar ground improvement methods would be a practical alternative for this project. The RAPs provide a dense/stiff vertical element with significant shearing resistance and will effectively increase the shear resistance within the zone that is being treated. RAPs also attract vertical loads from the overlying structure and distributes the load to the denser and stiffer layers beneath, thereby reducing total and differential settlement, which is an important consideration for large concrete water reservoirs. RAPs can also significantly reduce the risk of potential liquefaction-induced settlement by strengthening the zone being treated; however, the risk of liquefaction at this site is considered low.

RAPs are typically constructed by augering a shaft, typically 30 in. in diameter, to the bottom of the zone requiring improvement and backfilling the shaft with aggregate (crushed rock) that is compacted with a tamping ram in approximate 1-ft-thick lifts. RAPs are typically constructed using large hydraulic excavators equipped with augers and tampers. Augered RAP installation is generally limited to depths of 20 to 25 ft. An alternative method for RAP construction is installation using a hollow mandrel that is vibrated to the required depth instead of augered. Following insertion to the required depth, the mandrel is retracted as aggregate is placed in the bottom of the hole through the center of the mandrel. The mandrel is typically raised about 3 ft as the aggregate is placed and then driven back down about 2 ft to form a 1-ft-thick layer of compacted aggregate. Vibrated RAP methods can be used to construct RAPs to depths of up to 40 ft if conditions are favorable. Advantages of the vibratory RAP method are reduced spoils generation and it can be used in soft or loose soils below groundwater that may cave without casing.

To achieve the minimum required local seismic factor of safety, we recommend a minimum replacement ratio of about 30% (the ratio is the area of aggregate piers relative to the total area) using RAPs or comparable methods of ground improvement. For preliminary design purposes, it would be reasonable to assume the ground improvement footprint will be essentially square and need to extend at least 10 ft beyond the south half of the reservoir and 20 ft beyond the north half of the reservoir. The north side of the square treatment area should be parallel to the face of the slope north of the reservoir, which may require greater amounts of excavation than needed to construct the reservoir. It may be possible to limit the amount of excavation in the corner areas of the treatment area by using vibratory RAPs installed at or near existing grade. To provide adequate support for the RAP installation equipment and minimize the risk of subgrade disturbance, we recommend placing a minimum 18-in.-thick working blanket of compacted crushed rock over the reservoir subgrade. A greater thickness of crushed rock may be required if the subgrade is particularly soft. In this regard, the subgrade conditions should be evaluated by GRI before placing the working blanket. It is expected the working blanket will remain as part of the base course section beneath the reservoir. Recommendations for base course are discuss in the Foundation Support, Settlement, and Subdrainage section of this report.

As discussed above, construction of the RAPs using either a tamping foot or a vibrating mandrel to compact the aggregate backfill will result in ground vibrations. Based on our experience with similar projects that included RAP installation, vibrations from construction of RAPs typically decrease significantly over relatively short distances. Based on previous experience we do not anticipate adjacent residences will be subjected to vibrations in excess of currently acceptable construction levels. However, in our opinion, it would be prudent to install vibration instrumentation along the property lines of the site to monitor potential vibrations from construction equipment. Modifications can be made to construction procedures to reduce excessive vibrations, if necessary. Pre- and post-surveys of adjacent structures/residences should also be completed as part of the vibration monitoring program.

Site Preparation

Vegetation, roots, and other deleterious materials will not be suitable for use as structural fill; therefore, it will be necessary to remove surface organics prior to excavating soils that will be used later for structural fill. The ground surface in areas to receive new fills should also be stripped. Strippings may be used for landscaping purposes or should be removed from the site. We anticipate stripping to a depth of about 3 to 4 in. will be required in areas of lawn. Deeper stripping and grubbing will be required to remove brush and tree stumps where present. With the exception of backfilling around the new reservoir, we anticipate most soil that is excavated to complete the project will be removed from the site. However, stripped areas to receive structural fill should be evaluated by a qualified geotechnical engineer. Excavation spoils should not be stockpiled during construction within 75 ft of the slope along the north side of the site. The planned locations of soil stockpiles should be evaluated by GRI.

All concrete, piping, and other structural elements associated with the existing reservoir should be removed within the footprint of the new reservoir. Soft, loose, or otherwise unsuitable materials beneath the existing reservoir and within the footprint of the new reservoir should also be removed.

The fine-grained soils and decomposed basalt that mantle the site are sensitive to moisture content and are easily disturbed and softened by construction activity during wet conditions. In Addition, groundwater and site drainage, which are important for maintaining satisfactory slope stability during construction, will be more straightforward to manage during dry conditions. Therefore, we recommend as much site preparation and earthwork as practical be accomplished during the dry, summer months. It has been our experience that the moisture content of the upper approximate 2 to 3 ft of the silt will decrease during warm, dry weather. However, the moisture content of the soil below this depth tends to remain relatively unchanged and well above the optimum moisture content for compaction. As a result, the contractor must employ working procedures that prevent disturbance and softening of the subgrade soils. For this reason, excavation within the final 2 to 3 ft of subgrades should be accomplished with a trackhoe equipped with a smooth-edge bucket. It may be necessary to construct granular haul roads and work pads to provide access during wet conditions to minimize subgrade disturbance during construction. In general, a minimum 18- to 24-in. thickness of relatively clean, fragmental rock having a nominal maximum size of 4 to 6 in. would be required to support heavy construction traffic and protect the silt subgrade during wet ground conditions. If the subgrade is particularly soft, it may be prudent to place a geotextile fabric (AMOCO 2002, or equivalent) on the subgrade as a separation membrane prior to placing and compacting the granular work pad.

Excavation

General. Construction of the new reservoir will require an excavation of about 30 ft below existing site grades. The finished floor of the reservoir will be at about elevation 425 ft, and the bottom of the excavation will be at least 3 ft lower to accommodate the granular base course and subdrainage section. We anticipate the soils within the zone of excavation can be readily excavated with conventional excavation equipment, such as a large hydraulic trackhoe. The finished subgrade should be completed with a smooth-edge bucket as previously discussed. We anticipate significant portions of the reservoir will be established in the underlying predominantly decomposed to decomposed basalt. The borings made for this investigation indicate the basalt within the planned depth of excavation has a relative consistency comparable to medium stiff to stiff, fine-grained soil. Although not encountered in the borings, it is possible that zones of harder basalt and/or cobble- to boulder-size pieces of relatively hard basalt could be present within the depth of the excavation. The contractor should have means and methods available to accommodate excavation of potentially harder rock.

Cut Slopes. We recommend the temporary cut slopes made to construct the reservoir be no steeper than 1H:1V. However, flatter slopes maybe necessary to maintain an acceptable level of stability depending on the actual conditions exposed during construction, particularly in locations of groundwater seepage, if encountered in excavations. In this regard, temporary excavation slopes should be evaluated by a qualified geotechnical engineer at the time of construction.

Temporary slopes should be covered with plastic sheeting to reduce erosion during wet weather. In addition, excavation spoils and construction materials should not be stockpiled within 15 ft of the top of the temporary cut slope. The temporary excavation slopes should be evaluated on a daily basis by a knowledgeable person for obvious indications of slope instability such as sloughing, slumping, or ground cracks. Any indications of instability should be reported promptly to GRI for our evaluation. To minimize the risk of instability of temporary cut slopes, we recommend backfilling the reservoir excavation as soon as practical.

Depending on the time of year, perched groundwater may be present within the depth of excavation required to construct the reservoir. We anticipate that seepage, if encountered, can be controlled by pumping from sumps. A ditch should be installed at the top of the cut slopes to direct surface runoff away from the excavation. Water removed from the excavation should not be discharged on or near the top of the slope on the north site.

If temporary excavation slopes extend below the groundwater table or perched groundwater, a 6- to 12-in.-thick layer of relatively clean, well-graded crushed rock placed on the slopes may be required to reduce the risk of running soil conditions.

Permanent cut slopes following final grading, if present, should be no steeper than 2H:1V. Flatter cut slopes may be required if soft and/or wet ground conditions are encountered, which may also require installation of drainage. Permanent excavation slopes should be evaluated by a qualified geotechnical engineer at the time of construction so modifications can be made if necessary.

Temporary Shoring

As discussed previously, the side slopes of the excavation for the reservoir will be sloped at up to 1H:1V where space allows. However, we anticipate a shoring system constructed top-down, such as a tied-back soldier pile wall or possibly a soil-nail wall, may be necessary to retain the sides of the temporary excavation next to the existing pump station southeast of the planned reservoir and along the west side of the reservoir footprint near the properly line. The shoring could have a retained height of up to 30 ft. GRI can provide more detailed design and construction criteria for practical types of top-down shoring once detailed grading plans become available.

Structural Fill

As currently planned, backfill will be placed to within about 5 to 10 ft of the top of the reservoir. It is anticipated the backfill will consist of soil and/or decomposed basalt removed from excavations made during construction. With the exception of the tank backfill, no other significant fills are planned.

Excluding the surface strippings, excavation spoils approved by the geotechnical engineer may be used to backfill the reservoir. However, the fine-grained and decomposed basalt excavation spoils will be sensitive to moisture content and can only be placed and compacted during dry weather. Our investigation indicates the natural moisture content of the excavated materials will typically be in the range from 35 to 50%. In this regard, we anticipate the excavation spoils will require significant moisture conditioning and frequent field evaluations to confirm the material is being adequately compacted. If wet conditions prevent proper moisture conditioning of the excavation spoils, material used to construct structural backfills should consist of relatively clean, granular materials, such as sand, sandy gravel, or crushed rock. The maximum particle size of granular material placed against structures should be limited to not more than 1½ in. in diameter unless approved by the designer. A drainage blanket should be placed between common backfill and the side of embedded structures as discussed in the Lateral and Vertical Earth Pressures section of this report.

The structural backfill should be placed in horizontal lifts and compacted to at least 95% of the maximum dry density as determined by ASTM D 698 (standard Proctor). Fill placed within 5 ft of the reservoir should be compacted to 93 to 95% of the maximum dry density as determined by ASTM D 698 (standard Proctor) with small, light-weight compactors to avoid overcompaction and prevent the development of excessive lateral pressures. Appropriate lift thickness will depend on the type of compaction equipment used and the type of material being placed. For hand-operated or small compactors, we recommend a maximum loose lift thickness of 8 in. For moderate- to heavy-weight compactors, we recommend a maximum loose lift thickness of 12 in.

Finished fill slopes can be slightly overbuilt and then trimmed back to final grade using a trackhoe with a smooth-edge bucket. A qualified geotechnical engineer should review the proposed placement of any fill and evaluate the subgrade prior to fill placement. The proposed compaction equipment should be reviewed by the design team prior to fill placement to evaluate loads on embedded walls.

Landscape fill should be compacted to at least 90% of the maximum dry density as determined by ASTM D 698. The moisture content of soils placed in landscaped areas is generally not critical, provided construction equipment can effectively handle the material. Landscape fill should be no steeper than 3H:1V.

Foundation Support, Settlement, and Subdrainage

Based on information provided by PSE, the new reservoir foundation will consist of a 24-in.-thick, reinforced mat slab. In our opinion, a mat slab is a suitable foundation system for accommodating potential deformations that may occur as a result of the design-level seismic event. The reservoir was preliminary designed to consist of a 9-in.-thick roof slab supported by a 24-in.-diameter, reinforced concrete interior columns placed on a 20.5-ft center-to-center spacing that are cast directly into the mat slab (i.e., no spread footings on the top of the mat slab). The 12-in.-thick reservoir wall will also be cast directly into the mat slab. The maximum service (unfactored) loads are 90 kips for columns and 5.1 kips/ft for the wall, which do not include the weight of the water. A full reservoir of water will impose a uniform pressure of approximately 1,600 psf across the mat slab. Real bearing pressures of about 4,500 to 5,000 psf are estimated beneath the mat slab near column and wall locations for a full reservoir of water as the reservoir is currently configured.

To provide adequate support for the mat slab and assumed loading, we recommend the mat slab be underlain by a minimum 3-ft thickness of compacted crushed rock placed directly over the RAPs. The minimum 18-in.-thick working blanket placed for support of the RAP installation equipment can be considered part of the required base course section. However, it should be expected that the upper portion of the working blanket will be contaminated with soil and need to be removed. The amount of removal should be evaluated by the geotechnical engineer following RAP construction. Following removal, we recommend placing a subgrade geotextile prior to placing of remaining general granular base course and/or the assumed 2-ft-thick granular drainage layer discussed below.

General granular base course placed beneath the reservoir, including the RAP working blanket up to the bottom of the drainage layer, should consist of well-graded crushed rock with a maximum particle size of up to 1 1/2-in. meeting the requirements for Dense-Graded Aggregate as specified in Section 02630.10 of the Oregon Department of Transportation (ODOT) 2008 Standard Specifications for Highway Construction. The well-graded crushed rock should only be placed on firm, undisturbed subgrade that has been evaluated by a qualified geotechnical engineer. Soft or otherwise unsuitable materials that are identified at subgrade elevation should be overexcavated and replaced with granular structural fill. Other types of general granular material proposed by the contractor may be used with the approval of the design team. Materials used to construct drainage blankets should consist of open-graded, angular crushed rock with a maximum size of up to 1 1/2 in., with not more than about 2% passing the No. 200 sieve (washed analysis). Crushed rock of 3/4- to 1 1/2-in. gradation (drain rock) is commonly available and is suitable for this purpose. Open-graded rock (drain rock) placed on silty soil (where present) should be separated by a non-woven geotextile, such as Mirafi 140N or similar. All crushed rock placed beneath the reservoir should be compacted as structural fill using vibratory compaction equipment. The relative density of the well-graded compacted crushed rock should be at least 95% of the maximum dry density as determined by ASTM D 698 (standard Proctor). To protect the native subgrade soil, the initial lift of crushed rock base should be at least 12 in. thick. The drain rock cannot be density tested, but should be compacted until well keyed. The base course section (general granular base course plus drainage layer) should extend horizontally at least one-half the total thickness of the crushed rock section beyond the limits of the perimeter footing, or 1 1/2 ft for a 3-ft thickness of crushed rock.

RAP systems are typically designed by the RAP contractor to meet performance criteria developed by the reservoir designer. Based on similar reservoir projects with similar subsurface conditions, we anticipate RAPs installed to the harder decomposed basalt at depth of about 20 to 25 ft below the reservoir will limit total settlements (static condition) of the reservoir to about 3/4 to 1 1/4 in. when full of water and about one-half to two-thirds this amount near the edge of the reservoir, depending somewhat on the amount of fill placed on the sides of the reservoir. Further, we anticipate it should be feasible to limit differential settlements occurring between the edges of footings to a point on the floor slab halfway between any adjacent footings to a range of about 1/4 to 1/2 in. We do not anticipate any significant deformations will occur in the RAP-treated zone following the design-level earthquake.

For a subgrade prepared as discussed above and with the RAP-treated zone beneath the reservoir, we anticipate the mat slab for the reservoir can be designed to impose an allowable soil bearing pressure of up to 5,000 psf to limit settlements to the range of values discussed previously. We assume the 5,000 psf allowable bearing pressure will be used as performance criteria for the RAPs. This value applies to the total of dead load plus frequently and/or permanently applied live loads and can be increased by one-third for the total of all loads; dead, live, and wind or seismic. The allowable bearing pressure(s) and estimated settlements will need to be verified during design by the RAP designer

To address the actual deformation of the floor slab, we recommend analyzing the floor slab as a plate on an elastic foundation using a coefficient of subgrade reaction, k, of 100 pci. This value assumes the floor slab will be underlain by the aforementioned base course section above the RAP zone.

As discussed previously, the sides of the reservoir will be backfilled. Figure 2 indicates the backfill will extend up to about elevation 442 and 445 ft (17 to 20 ft thick) on the north and south side of the reservoir, respectively. We estimate these fills could induce up to 3/4 to 1 in. of settlement around the perimeter of the reservoir and should occur relatively quickly as the fill is placed. In our opinion, placement of the fill around the reservoir will not induce significant downdrag loads on the walls of the reservoir or settlement under the edge of the reservoir, assuming RAPs are installed beyond the edge of the reservoir as discussed previously.

Lateral loads (seismic, soil, etc.) can be resisted partially or completely by frictional forces developed between the base of the mat foundation and underlying crushed rock. The total frictional resistance between the mat slab and the underlying material is the normal force times the coefficient of friction between the crushed rock and the base of the reservoir. We recommend a value of 0.45 for the coefficient of friction between mass concrete cast directly on angular, granular structural fill. If a synthetic membrane, such as HDPE, is placed between the concrete and the underlying crushed rock, we recommend using a coefficient of friction of 0.30. If additional lateral resistance is required, passive earth pressures against embedded foundations and the reservoir walls can be computed on the basis of an equivalent fluid having a unit weight of 225 pcf for limiting lateral deflections to 1/4 to 1/2 in. and 300 pcf for larger deflections. These design passive earth pressures values would be applicable only if the backfill for the foundations or walls is placed as compacted structural fill where the backfill is horizontal. In areas where the backfill is sloped downward at 2H:1V these values should be reduced to about half. The coefficient of friction values provided above are also applicable for the frictional interaction of backfill soils against walls.

We anticipate perched groundwater could approach the ground surface and the bottom of the floor slab during periods of prolonged precipitation common from late fall through early spring. To limit hydrostatic forces on walls due to high groundwater and provide drainage for potential leakage through the reservoir floor slab, we recommend installing subdrainage beneath the floor slab of the new reservoir. We anticipate the reservoir will be underlain by a minimum 2-ft-thick layer of aforementioned open-graded crushed rock (drain rock) that will include 6-in.-diameter PVC drain pipes installed radially from the center of the reservoir in the lower part of the drainage layer outward to collection pipes at the perimeter of the reservoir. We recommend the radial drain pipes be spaced no greater than about 40 ft apart at the perimeter of the reservoir. The subdrainage section can be considered part of the recommended minimum 3-ft thickness of compacted crushed rock base course beneath the reservoir. The top 2 to 3 in. of the open-graded rock can be substituted with relatively clean 3/4-in.-minus crushed rock to facilitate leveling and placement of concrete.

Lateral Earth Pressures for Reservoir and Vaults

As discussed previously, the walls of the reservoir will be backfilled to within about 5 to 10 ft of the top of the reservoir. In addition, a valve vault embedded about 10 ft below site grades will also be constructed to service the new reservoir. Drainage will be provided on the sides and bottom of the reservoir to limit the risk of hydrostatic conditions from developing. We anticipate drainage will also be provided around valve vault. Lateral earth pressure and drainage recommendations for design of the reservoir and vault are provided below.

Design lateral earth pressures on embedded walls depend on the backfill geometry, drainage condition behind the wall, and the ability of the wall to yield by either translation or rotation away from the backfill. The two possible conditions regarding the ability of a wall to yield include the at-rest and the active earth pressure cases. The at-rest earth pressure case is applicable to a wall that is considered to be relatively rigid and unable to yield. The active earth pressure case is applicable to a wall that is capable of yielding slightly away from the backfill by either sliding or rotating about its base. A conventional cantilevered retaining wall is an example of a wall that develops the active earth pressure case by yielding. The walls of the new reservoir and valve vault will be braced at the top and bottom by the roof and floor and should be considered to be non-yielding. Yielding and non-yielding walls can be designed on the basis of a hydrostatic pressure based on an equivalent fluid having a unit weight of 35 and 55 pcf, respectively. In addition, it is assumed the backfill is fully drained and the surface of backfill is flat behind the wall.

We recommend using a distribution of 15 pcf to account for seismic earth pressures, with the resultant applied at 1/3H from the base of the structure, where H is the overall height of the soil retained. The seismic pressure should be added to the static earth pressures. Horizontal pressures due to surcharge loads, such as wheel loads associated with traffic on the backfill behind the walls, can be estimated using the guidelines provided on Figure 10. Transient surcharge loads, such as wheel loads, do not need to be included in the seismic loading case.

The backfill behind embedded walls must be fully drained for use of the aforementioned equivalent fluid values. The drainage system should consist of a minimum 2-ft-wide zone of free-draining granular fill adjacent to the embedded walls. The granular material used for the drainage layer behind embedded walls should conform to our previous recommendations for free-draining structural fill material. A 4- to 6-in.-diameter, rigid, perforated drain pipe should be provided near the bottom of the embedded wall. A non-woven geotextile, such as Mirafi 140N (or similar), is recommended between the free-draining backfill and the general wall backfill to reduce the risk of contamination of the wall drain system. Recommendations regarding placement of backfill behind embedded walls are provided in the Structural Fill section of this report.

Utilities

The project will include replacing the existing 18-in.-diameter inlet/outlet line in Skyline Circle with a 24-in.-diameter line and the existing 8-in.-diameter PVC main north of the reservoir with an 8-in.-diameter ductile iron line. A new overflow line will also be constructed and extend northward from the north side of the reservoir down the slope north of the site; the discharge location has not yet been determined. We anticipate subsurface drainage from the reservoir will likely be conveyed in piping to a point downslope of the reservoir.

We anticipate the maximum depth of trenches for installation of the piping will be 4 to 6 ft below the finished ground surface except where it connects to the new reservoir. Depending on the time of year, groundwater seepage could be encountered in utility excavations, which could create the potential for running soil conditions and unstable trench sidewalls. All excavation sidewalls should be properly sloped or shored to conform to applicable local, state, or federal regulations. Some overexcavation of the trench bottom may also be necessary to permit installation of stabilization/drainage material if wet ground conditions are encountered. To provide a relatively dry working base and facilitate dewatering, a drainage/stabilization layer consisting of a 12- to 18-in. thickness of open-graded crushed rock (drain rock) containing less than 2% passing the No. 200 sieve (washed analysis) may be appropriate. However, the need for a stabilization layer should be evaluated based on actual conditions. We anticipate that seepage, where encountered, can be controlled by pumping from sumps in the trench excavation.

Utility trenches beneath or near pavement, the reservoir foundation, sidewalks, slabs, other structures, should be backfilled with well-graded crushed rock with a maximum particle size of up to 1½-in. and meeting the requirements for Dense-Graded Aggregate as specified in Section 02630.10 of the ODOT 2008 Standard Specifications for Highway Construction. The crushed rock backfill should be compacted to at least 95% of the maximum dry density as determined by ASTM D 698 in the upper 4 ft of the trench and at least 92% of this density below this depth. The use of trackhoe-mounted vibratory plate compactors is usually most efficient for compaction of trench backfill. Lift thicknesses should be evaluated on the basis of field density tests; however, particular care should be taken when operating hoe-mounted compactors to prevent damage to the newly placed utilities. Flooding or jetting to compact the trench backfill should not be permitted.

Due to slope stability considerations, the backfill placed in utility trenches on the sloping ground north of the reservoir should be compacted to at least 92% maximum dry density as determined by ASTM D 698. In addition, it would also be prudent to install a 4-in.-diameter perforated drain pipe in the granular pipe bedding to collect any groundwater that may be intercepted during wet conditions. The perforated drain pipes should be discharged into a stormwater system and not discharge directly onto the slope.

Utility pipes should be underlain by a minimum 6-in. thickness of good-quality bedding material. We recommend the bedding material and any pipe zone backfill consist of relatively clean, granular material such as 3/4- or 1-in.-minus crushed rock. Material conforming to ODOT specifications for dense-graded aggregate would be suitable for this purpose. The bottom of the excavation should be thoroughly cleaned to remove loose materials before installing the bedding material.

Design Review and Construction Services

We welcome the opportunity to review and discuss construction plans and specifications for this project as they are being developed. In addition, GRI should be retained to review all geotechnical-related portions of the plans and specifications to evaluate whether they are in conformance with the recommendations provided in our report. In addition, to observe compliance with the intent of our recommendations, design concepts, and the plans and specifications, we are of the opinion that all construction operations dealing with earthwork and foundations should be observed by a GRI representative. Our construction-phase services will allow for timely design changes if site conditions are encountered that are different from those described in this report. If we do not have the opportunity to confirm our interpretations, assumptions, and analyses during construction, we cannot be responsible for the application of our recommendations to subsurface conditions that are different from those described in this report.

Submitted for GRI,

A. Wesley Spang, PhD, PE, GE Principal	Keith S. Martin, PE, GE Project Engineer	George Freitag, CEG Associate

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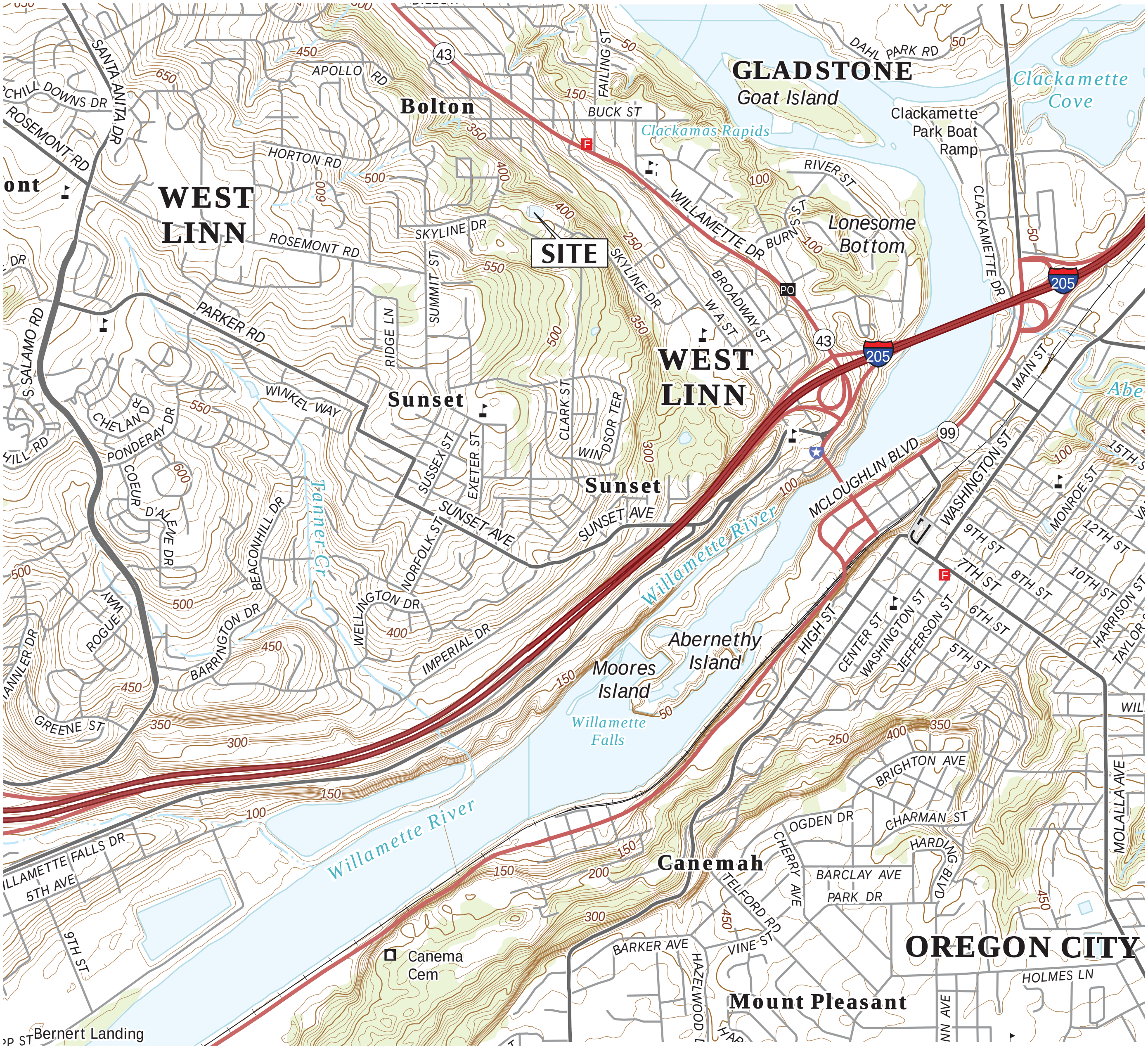

USGS TOPOGRAPHIC MAP
OREGON CITY, OREG. (2014)

North

0 1/2 1 MILE

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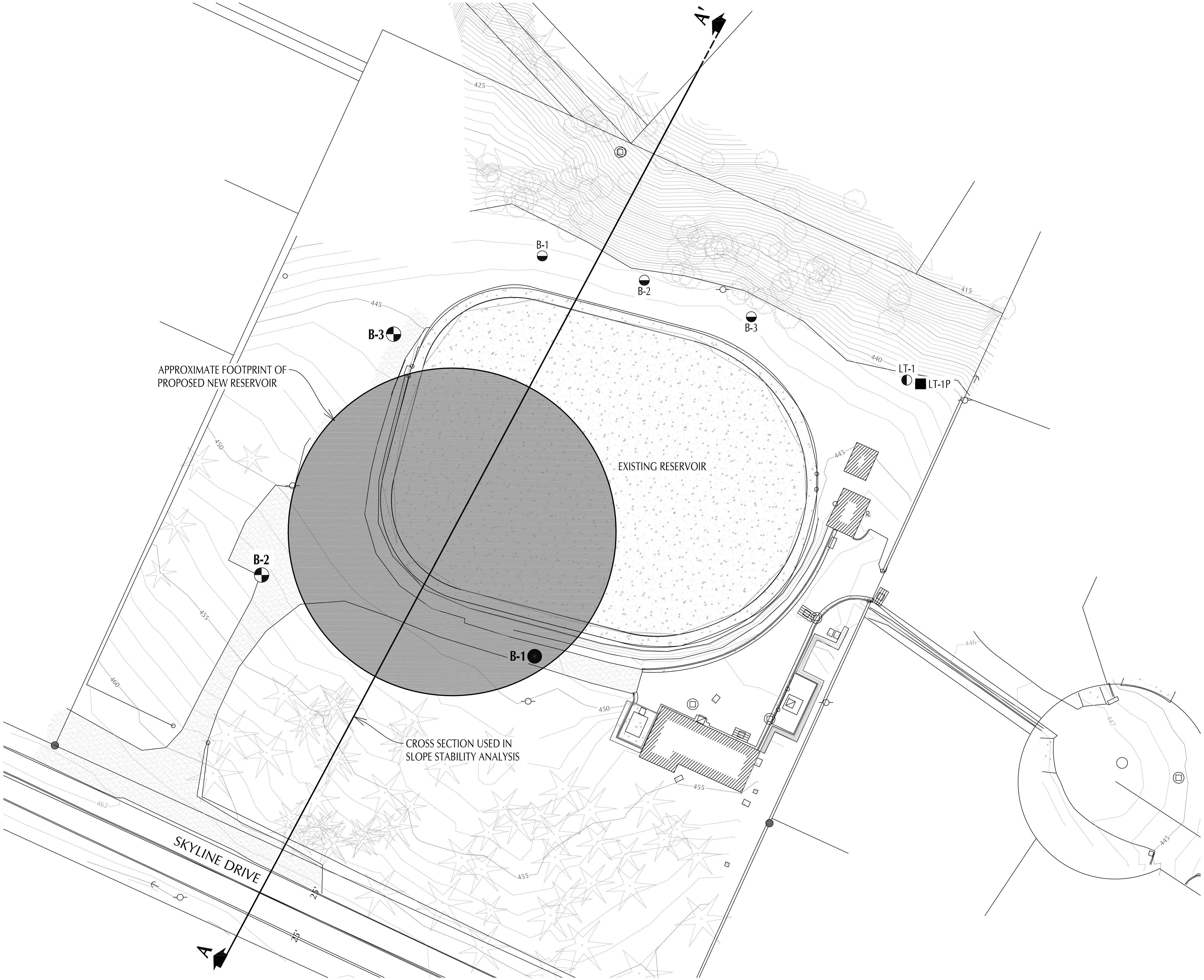
BOLTON RESERVOIR

VICINITY MAP

JAN. 2015 JOB NO. 5338-A FIG. 1



BORING MADE BY GRI
(NOVEMBER 27 - 29, 2014)

BORING MADE BY GRI
(JUNE 15, 2012)

BORING AND INCLINOMETER MADE / INSTALLED BY
LANDSLIDE TECHNOLOGY
(1997)

STANDPIPE INSTALLED BY LANDSLIDE TECHNOLOGY
(1997)

BORING MADE BY NORTHWEST TESTING LABORATORIES
(1972)

ELEVATION DATUM NAVD 88

SITE PLAN FROM FILE BY MURRAY, SMITH & ASSOCIATES, INC.

North

0 50 100 FT

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SITE PLAN

JAN. 2015 JOB NO. 5338-A FIG. 2

SITE MAP FROM AERIAL PHOTO BY BING IMAGE (UNDATED)

North

0 250 500 FT

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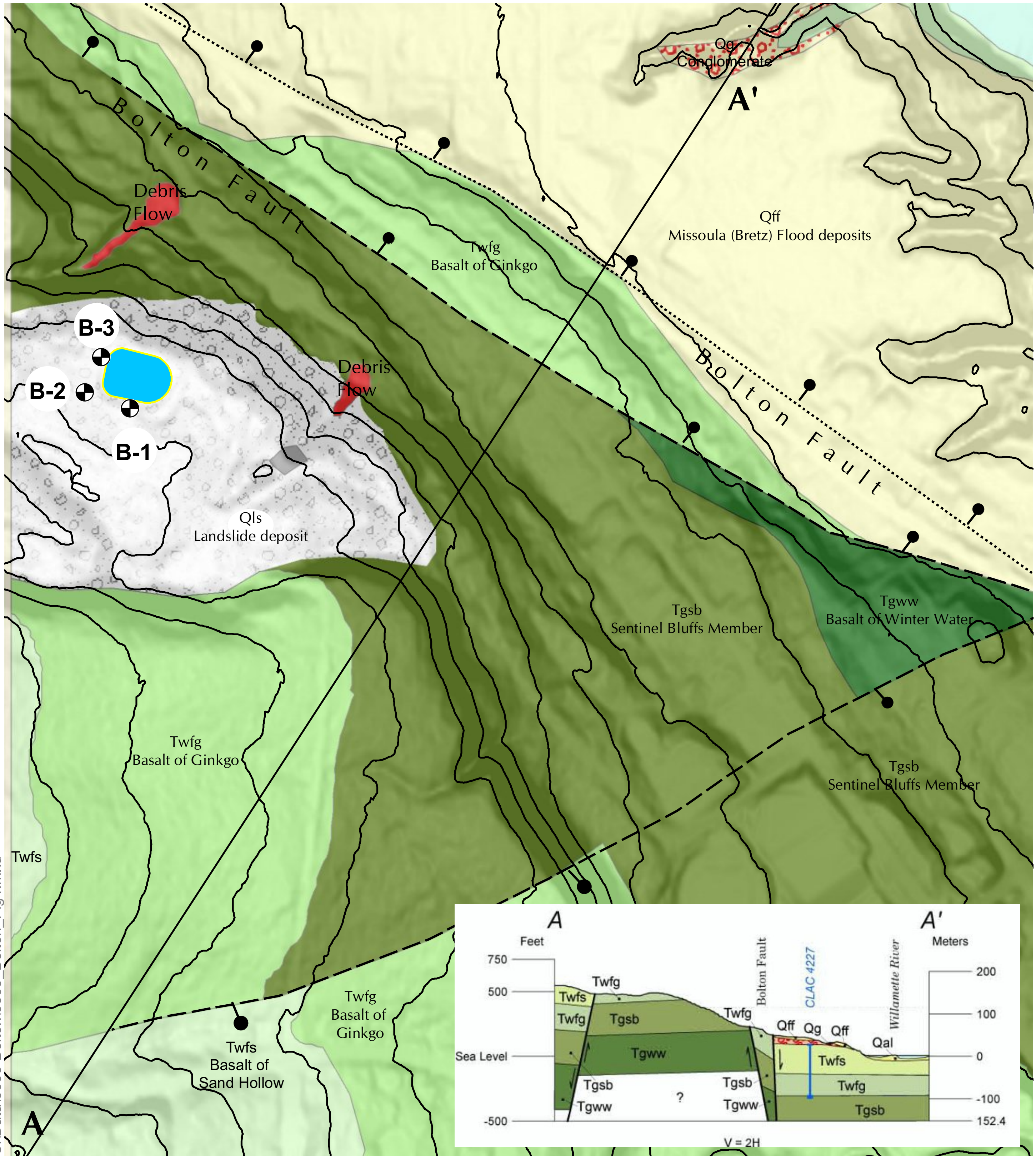
BOLTON RESERVOIR

SITE MAP

JAN. 2015 | JOB NO. 5338-A | FIG. 3



C:\Data\5338-Bolton\5338_Bolton_Fig4.mxd

Source Madin (2009)

Borings by GRI (2012, 2014)

Bolton Reservoir

North

0 300 600 Ft

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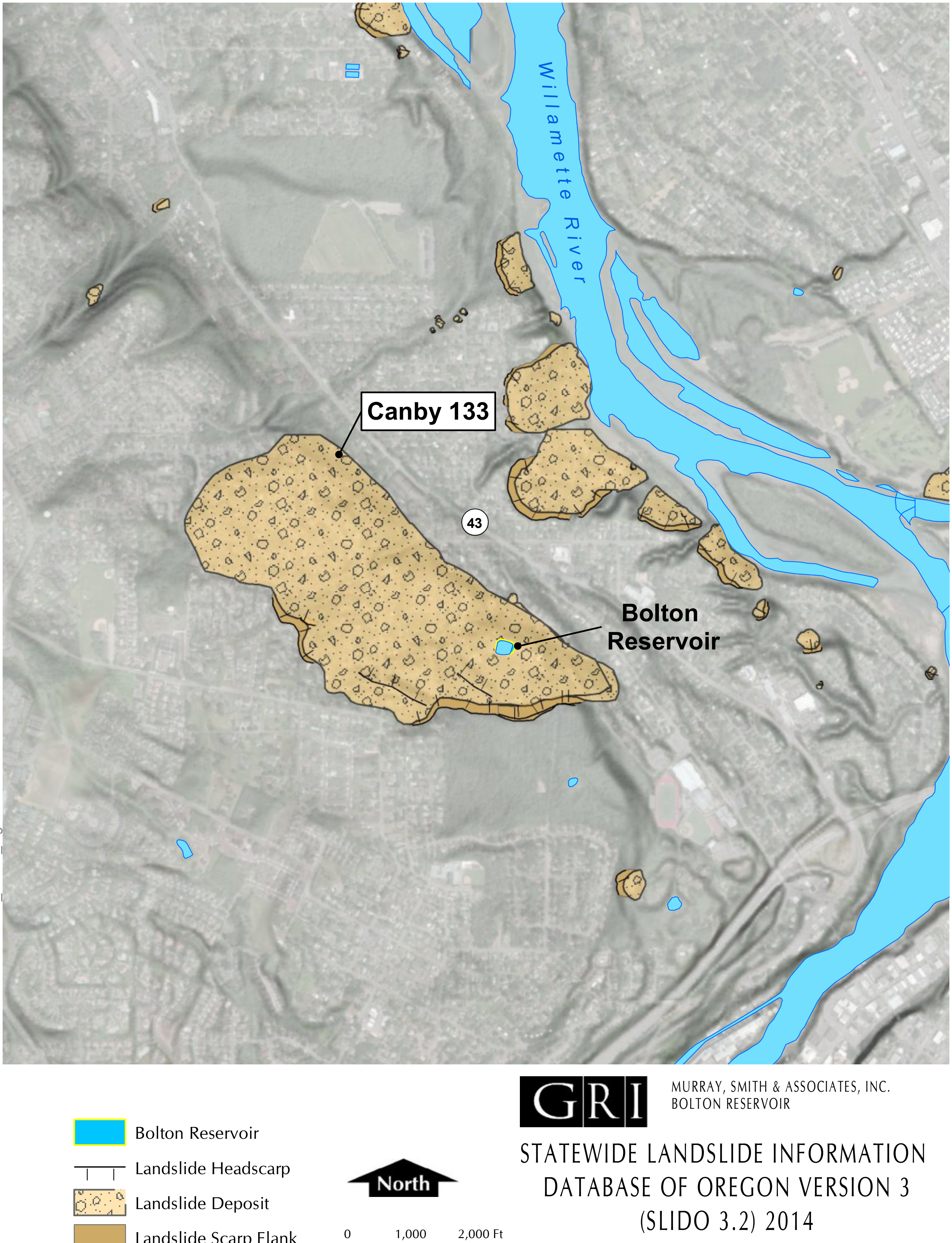
BOLTON RESERVOIR

GEOLOGIC MAP AND CROSS SECTION

JAN. 2015 JOB NO. 5338-A FIG.4



C:\Data\5338-Bolton\5338_Bolton_Fig5.mxd

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BOLTON RESERVOIR

STATEWIDE LANDSLIDE INFORMATION DATABASE OF OREGON VERSION 3 (SLIDO 3.2) 2014

JAN. 2015 | JOB NO. 5338-A | FIG.5

Material Properties:

Crushed Rock Base Course
Total Unit Wt. = 130 pcf
phi' = 40 degrees
c' = 0

Structural Backfill
Total Unit Wt. = 120 pcf
phi' = 32 degrees
c' = 0

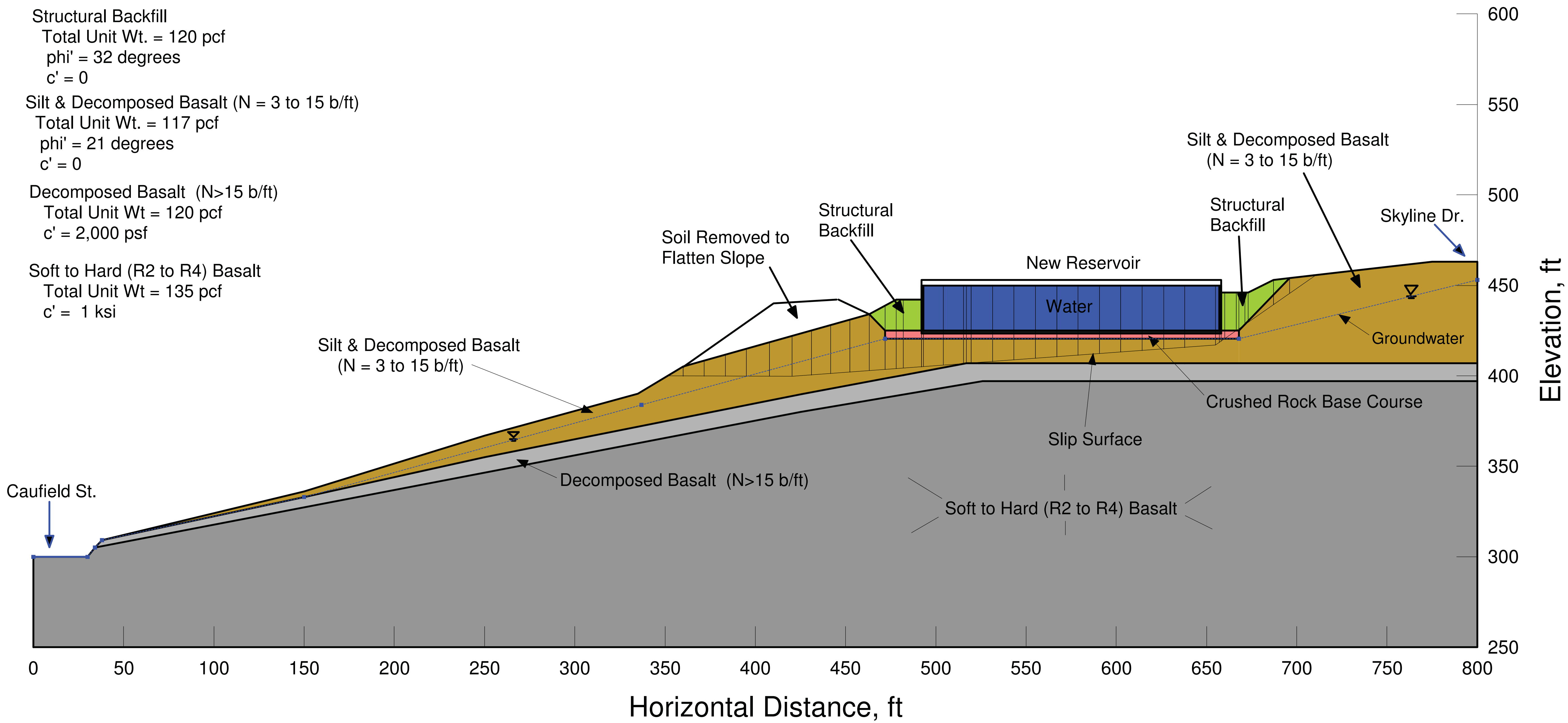
Silt & Decomposed Basalt (N = 3 to 15 b/ft)
Total Unit Wt. = 117 pcf
phi' = 21 degrees
c' = 0

Decomposed Basalt (N>15 b/ft)
Total Unit Wt = 120 pcf
c' = 2,000 psf

Soft to Hard (R2 to R4) Basalt
Total Unit Wt = 135 pcf
c' = 1 ksi

Minimum Factor of Safety Values:
Seismic = 1.0
Static > 1.5

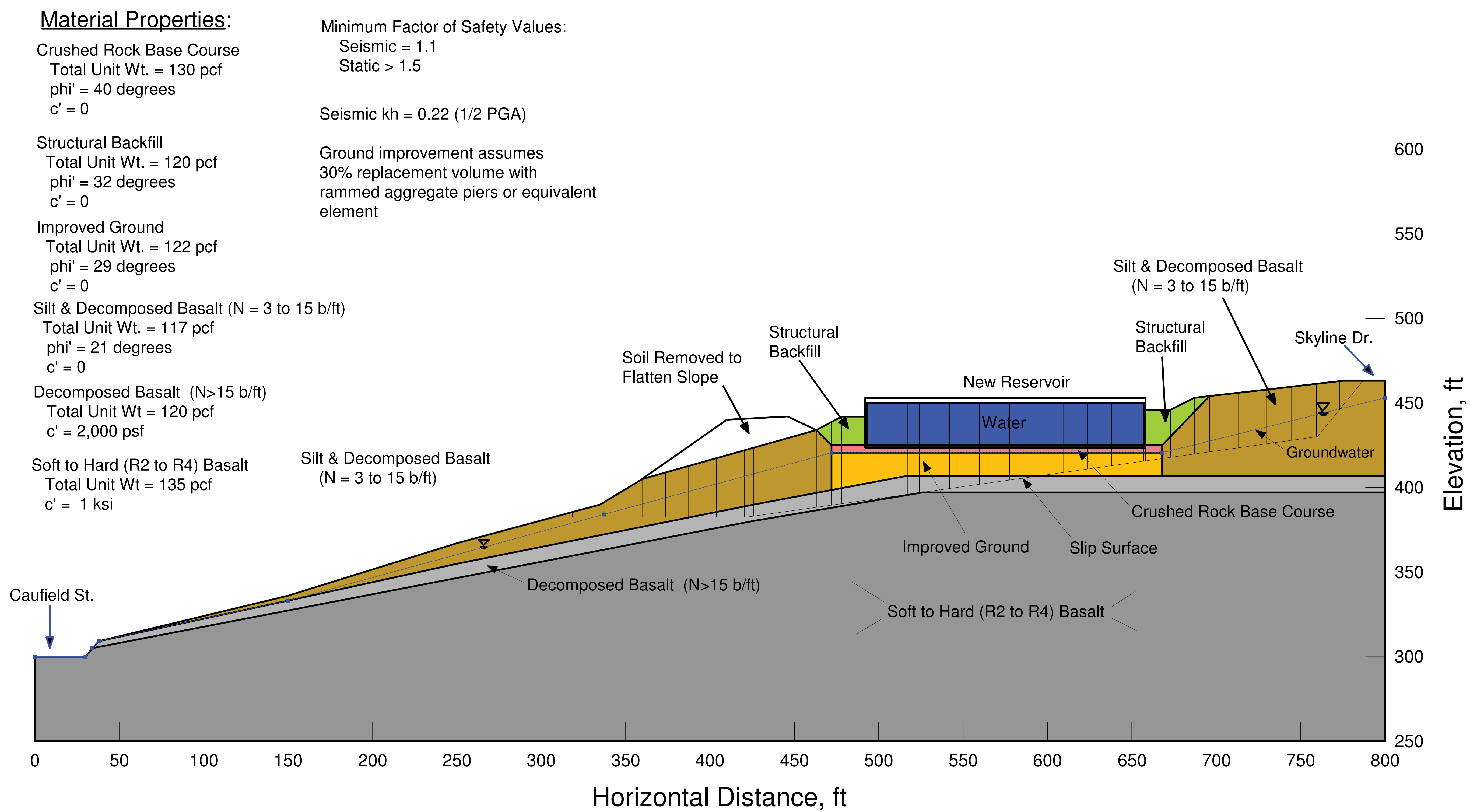
Seismic kh = 0.22 (1/2 PGA)



GRI MURRAY, SMITH & ASSOCIATES, INC.
BOLTON RESERVOIR

SLOPE STABILITY MODEL
(NEW RESERVOIR WITHOUT GROUND IMPROVEMENT)

JAN. 2015 JOB NO. 5338-A FIG. 6



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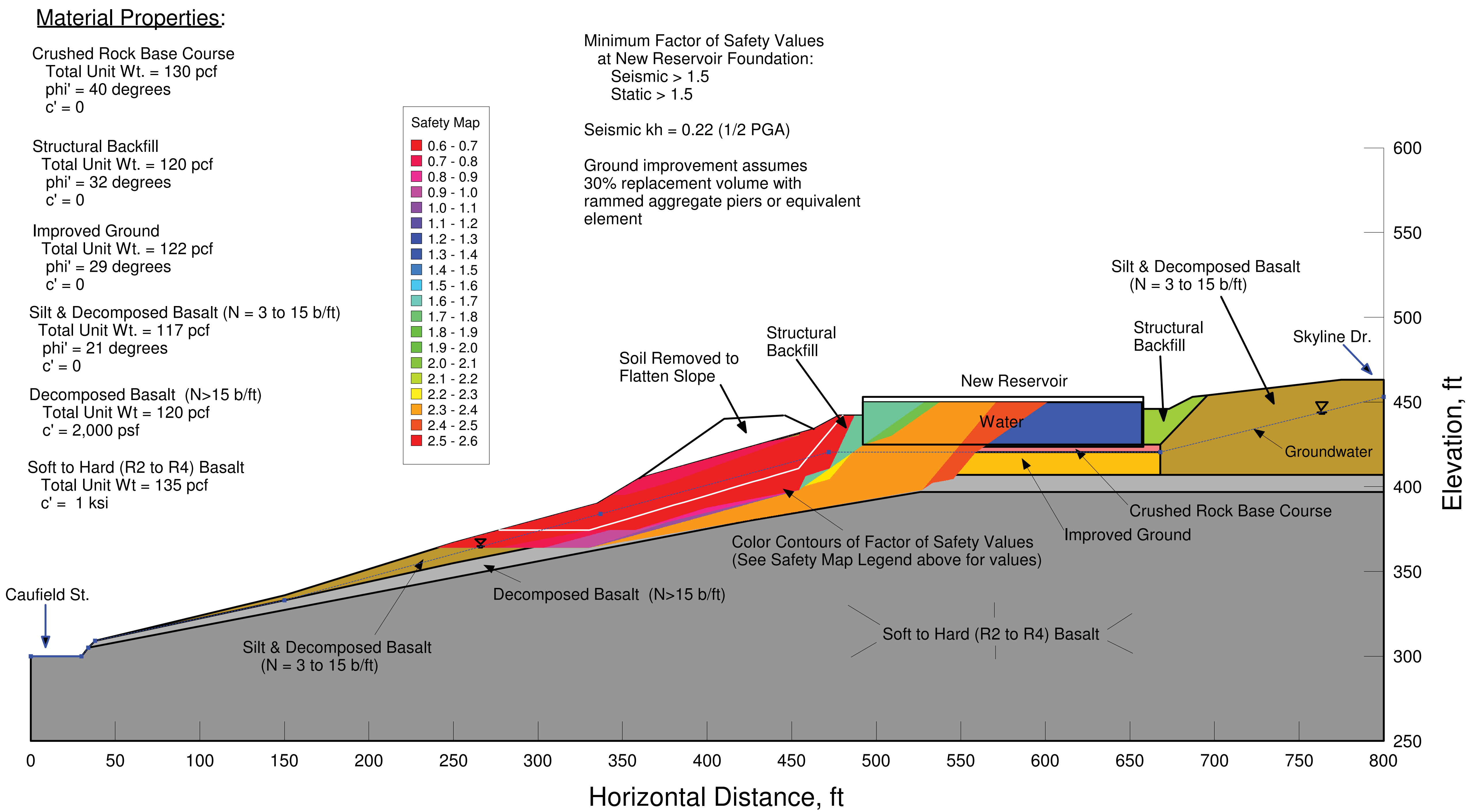
MURRAY, SMITH & ASSOCIATES, INC.
BOLTON RESERVOIR

SLOPE STABILITY MODEL
(NEW RESERVOIR WITH GROUND IMPROVEMENT)

JAN. 2015 | JOB NO. 5338-A | FIG. 7



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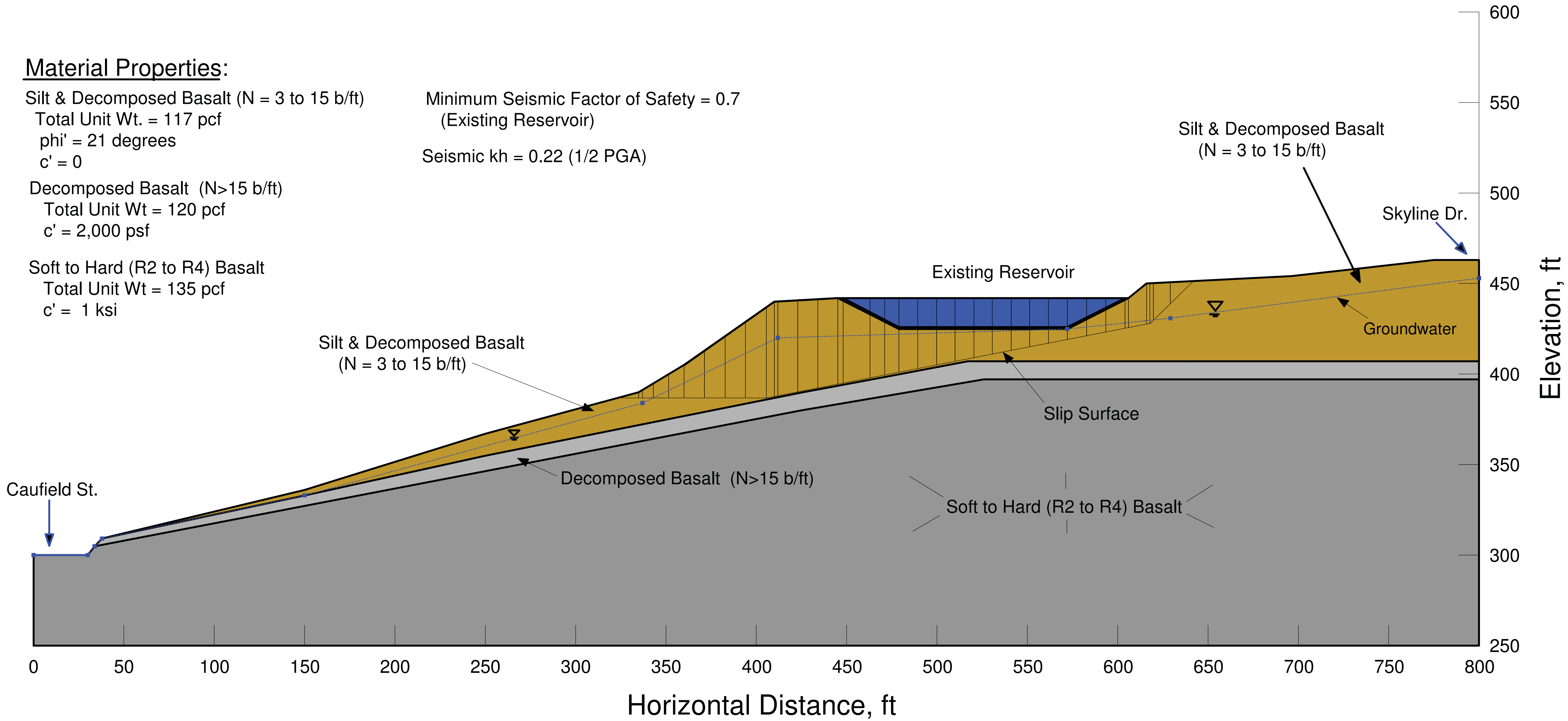
BOLTON RESERVOIR

SLOPE STABILITY MODEL
(NEW RESERVOIR WITH GROUND IMPROVEMENT, NORTH SLOPE)

JAN. 2015 JOB NO. 5338-A FIG. 8



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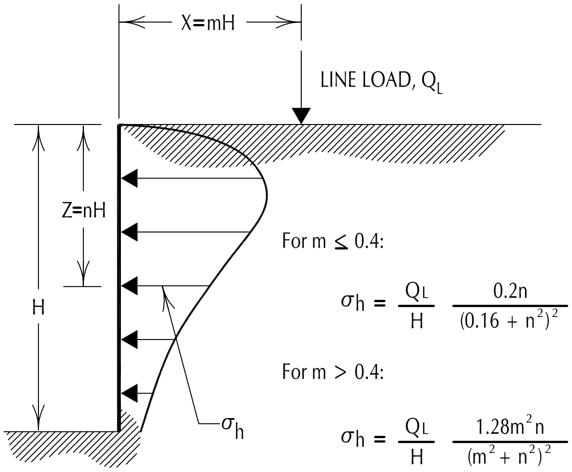
MURRAY, SMITH & ASSOCIATES, INC.
BOLTON RESERVOIR

SLOPE STABILITY MODEL
(EXISTING RESERVOIR)

JAN. 2015 | JOB NO. 5338-A | FIG. 9



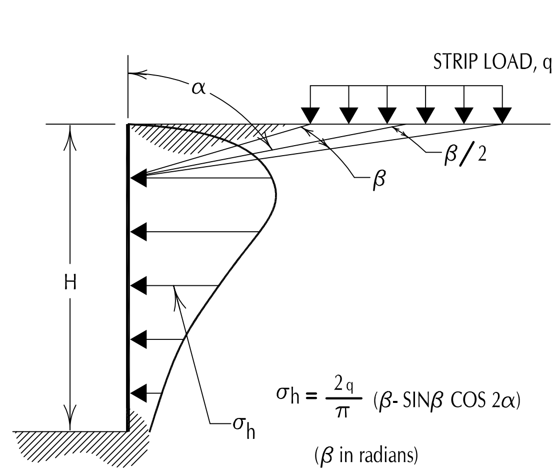
For m ≤ 0.4:

$$\sigma_h = \frac{Q_L}{H} \frac{0.2n}{(0.16 + n^2)^2}$$

For m > 0.4:

$$\sigma_h = \frac{Q_L}{H} \frac{1.28m^2 n}{(m^2 + n^2)^2}$$

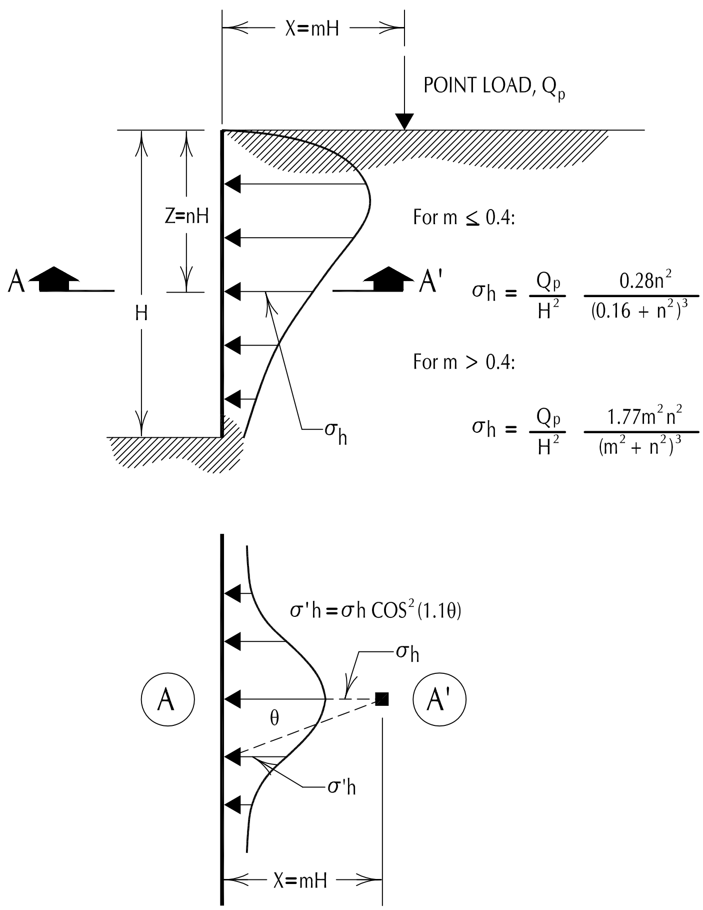
LINE LOAD PARALLEL TO WALL



$$\sigma_h = \frac{2q}{\pi} (\beta - \text{SIN}\beta \ \text{COS } 2\alpha)$$

(β in radians)

STRIP LOAD PARALLEL TO WALL



For m ≤ 0.4:

$$\sigma_h = \frac{Q_p}{H^2} \frac{0.28n^2}{(0.16 + n^2)^3}$$

For m > 0.4:

$$\sigma_h = \frac{Q_p}{H^2} \frac{1.77m^2 n^2}{(m^2 + n^2)^3}$$

DISTRIBUTION OF HORIZONTAL PRESSURES

VERTICAL POINT LOAD

NOTES:

1. THESE GUIDELINES APPLY TO RIGID WALLS WITH POISSON'S RATIO ASSUMED TO BE 0.5 FOR BACKFILL MATERIALS.

2. LATERAL PRESSURES FROM ANY COMBINATION OF ABOVE LOADS MAY BE DETERMINED BY THE PRINCIPLE OF SUPERPOSITION.

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BOLTON RESERVOIR

SURCHARGE-INDUCED LATERAL PRESSURE

JAN. 2015 | JOB NO. 5338-A | FIG. 10

APPENDIX A

Field Explorations and Laboratory Testing

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATIONS

General

Subsurface materials and conditions at the site were evaluated by GRI on June 15, 2012, with one boring designated B-1, and on October 27 through 29, 2014, with two borings, designated B-2 and B-3. The locations of the borings are shown on Figure 2. All explorations were observed by a certified engineering geologist from GRI.

The borings were advanced to depths ranging from 76 to 90 ft with mud-rotary drilling methods using CME 75 track- and truck-mounted drill rigs provided and operated by Western States Soil Conservation, Inc., of Hubbard, Oregon. Disturbed and undisturbed samples were obtained from the borings at about 2.5- to 5-ft intervals of depth. Disturbed samples were obtained using a standard split-spoon sampler. At the time of sampling, the Standard Penetration Test was conducted. This test consists of driving a standard split-spoon sampler into the soil a distance of 18 in. using a 140-lb hammer dropped 30 in. The number of blows required to drive the sampler the last 12 in. is known as the Standard Penetration Resistance, or N-value. The N-values provide a measure of the relative density of granular soils and the relative consistency of cohesive soils. The soil and rock samples obtained in the split-spoon sampler were carefully examined in the field, and representative portions were saved in airtight jars for further examination and physical testing in our laboratory. In addition, relatively undisturbed Shelby tube samples of soil and decomposed rock were collected and returned to our laboratory for further evaluation and testing. Below a depth of about 64and 60 ft in boring B-1 and B-2, respectively, and 55 ft in boring B-3 wireline coring methods were used to obtain continuous samples of rock. The rock cores were placed in core boxes and returned to our laboratory for further evaluation.

Logs of the borings are provided on Figures 1A through 3A. Each log presents a descriptive summary of the various types of materials encountered in the boring and notes the depth where the materials and/or characteristics of the materials change. To the right of the descriptive summary, the numbers and types of samples taken during the drilling operation are indicated. Farther to the right, N-values are shown graphically, along with the natural moisture contents, Torvane shear strength values, Atterberg limits, and percentage of material passing the No. 200 sieve. The terms and symbols used to describe the soil and rock encountered in the borings are defined in Tables 1A and 2A and the attached legend.

Observation Standpipe

An observation standpipe piezometer was installed in boring B-2 and B-3 to depths of about 90 and 48 ft, respectively. The standpipes consist of a 1-in.-I.D. plastic pipe slotted below a depth of 60 and 17 ft in boring B-2 and B-3, respectively. Each boring was flushed with clean water prior to installing the pipe, and the annular space around the pipe was backfilled with Colorado Sand to about 1 ft above the slotted zone. The remaining portion of the hole was backfilled with a seal consisting of bentonite. The top of the standpipe is protected with a flush-mounted monument. Groundwater enters through the slots and rises to a static level, which is measured with an electrical probe lowered inside the pipe.

LABORATORY TESTING

General

The samples obtained from the borings were examined in our laboratory, where the physical characteristics of the samples were noted, and the field classifications were modified where necessary. At the time of classification, the natural moisture content of each sample was determined. Additional tests included determinations of Torvane shear strengths, undisturbed unit weights, one-dimensional consolidation testing, washed sieve analysis, Atterberg limits, drained residual torsional shear strength, and grain-size analysis.

Natural Moisture Contents

Natural moisture content determinations were made in conformance with ASTM D 2216. The results are summarized on the Boring Logs, Figures 1A through 3A.

Torvane Shear Strength

The approximate undrained shear strength of the fine-grained soils obtained in the Shelby tubes was measured using the Torvane shear device. The Torvane is a hand-held apparatus with vanes that are inserted into the soil. The torque required to fail the soil in undrained shear around the vanes is measured using a calibrated spring. The torque measurements have been correlated to the undrained shear strength of various fine-grained soils. The results of the Torvane shear strength testing are shown on Figures 1A through 3A.

Undisturbed Unit Weight

The dry unit weight, or dry density, of undisturbed soil samples was determined in the laboratory in substantial conformance with ASTM D 2937. The unit weight determinations are summarized below.

SUMMARY OF UNIT WEIGHT DETERMINATIONS

Boring	Sample	Approximate Depth, ft	Soil Type	Moisture Content, %	Dry Unit Weight, pcf
B-1	S-2	8.2	Clayey SILT, some fine- to medium-grained sand, brown, stiff (Landslide Debris)	40	81.7
	S-5	16.2	Clayey SILT, some fine- to medium-grained sand, brown, stiff (Landslide Debris)	31	94.3
	S-10	35.7	Clayey SILT, trace sand- to gravel-size fragments of extremely soft (R0), predominantly decomposed basalt, stiff to very stiff (Landslide Debris)	37	88.0
B-2	S-4	11.3	SILT, some clay to clayey, trace to some fine-grained sand, red-brown, black manganese staining, medium stiff (Landslide Debris)	35	87.8
	S-8	21.2	Clayey SILT, trace to some fine-grained sand, brown to red-brown, stiff (Landslide Debris)	27	101.5
	S-11	31.3	BASALT, gray-brown, decomposed, extremely soft (R0), manganese oxide mineralization, relic rock structure, consistency of medium stiff soil (Wanapum Basalt; Landslide Debris)	44	80.0
	S-14	37.8	BASALT, gray-brown to red-brown, decomposed, extremely soft (R0), manganese oxide mineralization, relic rock structure, consistency of soft to hard soil (Wanapum Basalt; Landslide Debris)	43	76.0

Boring	Sample	Approximate Depth, ft	Soil Type	Moisture Content, %	Dry Unit Weight, pcf
B-2	S-16	46.8	BASALT, gray-brown to red-brown, decomposed, extremely soft (R0), manganese oxide mineralization, relic rock structure, consistency of soft to hard soil (Wanapum Basalt; Landslide Debris)	39	84.0
B-3	S-6	15.8	BASALT, gray-brown to red-brown, decomposed, extremely soft (R0), secondary mineralization, relic rock structure, consistency of soft to hard soil (Wanapum Basalt; Landslide Debris)	49	76.0
	S-10	26.0	BASALT, gray-brown to red-brown, decomposed, extremely soft (R0), secondary mineralization, relic rock structure, consistency of soft to hard soil (Wanapum Basalt; Landslide Debris)	52	68.0

One-Dimensional Consolidation Testing

Two, one-dimensional consolidation test was performed in conformance with ASTM D 2435 on relatively undisturbed samples from borings B-1 and B-2 at a depth of about 16.5 and 37.3 ft, respectively. The test provides data on the compressibility of the underlying fine-grained soils and decomposed rock, necessary for settlement studies. The test results are summarized on Figures 4A and 5A in the form of a curve showing percent strain versus applied effective stress. The initial dry unit weight and moisture content of the samples are also shown on the figures.

Washed-Sieve Analysis

Washed sieve analyses were performed using selected soil samples to assist in classification of the soils. The test is performed by taking a sample of known dry weight and washing it over a No. 200 sieve. The material retained on the sieve is oven-dried and weighed. The percentage of material passing the No. 200 sieve is then calculated. The results are tabulated below and shown on Figures 2A and 3A.

SUMMARY OF WASHED SIEVE ANALYSES

Boring	Sample	Depth, ft	Percent Passing No. 200 Sieve	Description
B-2	S-6	15.0	90	Clayey SILT, some fine-grained sand, brown to reddish-brown, stiff (Landslide Debris)
	S-7	17.5	90	Clayey SILT, some fine-grained sand, brown to reddish-brown, stiff (Landslide Debris
	S-9	22.0	85	Clayey SILT, some fine-grained sand, brown to reddish-brown, stiff (Landslide Debris)
B-3	S-4	10.0	82	Clayey SILT, some fine-grained sand, brown (Landslide Debris)

Atterberg Limits

Atterberg limits determinations were performed by GRI on representative samples in conformance with ASTM D 4318. The results of the tests completed by GRI are summarized on Figure 6A Atterberg limits testing were also performed by Cooper Testing Laboratory of Palo Alto, California, on a representative sample decomposed basalt from a depth of 35 ft in boring B-2 that was used to perform the drained residual torsional shear strength test discussed below. The results of the Atterberg limit test by Cooper Testing Laboratory are shown on Figure 7A.

Drained Residual Torsional Shear Strength

The drained residual torsional shear strength test of a representative sample of decomposed basalt from a depth of 35 ft in boring B-2 was completed in conformance with ASTM D 6467 by Cooper Testing Laboratory. The results of the test are summarized on Figure 8A.

Grain Size Analysis

Grain size analysis was completed by Cooper Testing Laboratory of Palo Alto, California on representative sample decomposed basalt from a depth of 35 ft in boring B-2 that was used to perform the drained residual torsional shear strength test discussed above in conformance with ASTM D 422. The results of the test are shown on Figure 9A.

Table 1A

GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil

Relative Density	Standard Penetration Resistance (N-values) blows per foot
very loose	0 - 4
loose	4 - 10
medium dense	10 - 30
dense	30 - 50
very dense	over 50

Description of Consistency for Fine-Grained (Cohesive) Soils

Consistency	Standard Penetration Resistance (N-values) blows per foot	Torvane or Undrained Shear Strength, tsf
very soft	0 - 2	less than 0.125
soft	2 - 4	0.125 - 0.25
medium stiff	4 - 8	0.25 - 0.50
stiff	8 - 15	0.50 - 1.0
very stiff	15 - 30	1.0 - 2.0
hard	over 30	over 2.0

Grain-Size Classification

Boulders:
- $>$12 in.

Cobbles:
- 3 - 12 in.

Gravel:
- 1/4 - 3/4 in. (fine)
- 3/4 - 3 in. (coarse)

Sand:
- No. 200 - No. 40 sieve (fine)
- No. 40 - No. 10 sieve (medium)
- No. 10 - No. 4 sieve (coarse)

Silt/Clay:
- pass No. 200 sieve

Modifier for Subclassification

Adjective	Primary Constituent SAND or GRAVEL	Primary Constituent SILT or CLAY
	Percentage of Other Material (by weight)	
trace:	5 - 15 (sand, gravel)	5 - 15 (sand, gravel)
some:	15 - 30 (sand, gravel)	15 - 30 (sand, gravel)
sandy, gravelly:	30 - 50 (sand, gravel)	30 - 50 (sand, gravel)
trace:	$<$5 (silt, clay)	*Relationship of clay and silt determined by plasticity index test*
some:	5 - 12 (silt, clay)	
silty, clayey:	12 - 50 (silt, clay)	

Table 2A: GUIDELINES FOR CLASSIFICATION OF ROCK

RELATIVE ROCK WEATHERING SCALE

Term	Field Identification
Fresh	Crystals are bright. Discontinuities may show some minor surface staining. No discoloration in rock fabric.
Slightly Weathered	Rock mass is generally fresh. Discontinuities are stained and may contain clay. Some discoloration in rock fabric. Decomposition extends up to 1 in. into rock.
Moderately Weathered	Rock mass is decomposed 50% or less. Significant portions of rock show discoloration and weathering effects. Crystals are dull and show visible chemical alteration. Discontinuities are stained and may contain secondary mineral deposits.
Predominantly Decomposed	Rock mass is more than 50% decomposed. Rock can be excavated with geologist's pick. All discontinuities exhibit secondary mineralization. Complete discoloration of rock fabric. Surface of core is friable and usually pitted due to washing out of highly altered minerals by drilling water.
Decomposed	Rock mass is completely decomposed. Original rock "fabric" may be evident. May be reduced to soil with hand pressure.

RELATIVE ROCK HARDNESS SCALE

Term	Hardness Designation	Field Identification	Approximate Unconfined Compressive Strength
Extremely Soft	R0	Can be indented with difficulty by thumbnail. May be moldable or friable with finger pressure.	< 100 psi
Very Soft	R1	Crumbles under firm blows with point of a geology pick. Can be peeled by a pocket knife and scratched with fingernail.	100 - 1,000 psi
Soft	R2	Can be peeled by a pocket knife with difficulty. Cannot be scratched with fingernail. Shallow indentation made by firm blow of geology pick.	1,000 - 4,000 psi
Medium Hard	R3	Can be scratched by knife or pick. Specimen can be fractured with a single firm blow of hammer/geology pick.	4,000 - 8,000 psi
Hard	R4	Can be scratched with knife or pick only with difficulty. Several hard hammer blows required to fracture specimen.	8,000 - 16,000 psi
Very Hard	R5	Cannot be scratched by knife or sharp pick. Specimen requires many blows of hammer to fracture or chip. Hammer rebounds after impact.	> 16,000 psi

RQD AND ROCK QUALITY

Relation of RQD and Rock Quality

RQD (Rock Quality Designation), %	Description of Rock Quality
0 - 25	Very Poor
25 - 50	Poor
50 - 75	Fair
75 - 90	Good
90 - 100	Excellent

Terminology for Planar Surface

Bedding	Joints and Fractures	Spacing
Laminated	Very Close	< 2 in.
Thin	Close	2 in. – 12 in.
Medium	Moderately Close	12 in. – 36 in.
Thick	Wide	36 in. – 10 ft
Massive	Very Wide	> 10 ft

BORING AND TEST PIT LOG LEGEND

SOIL SYMBOLS

Symbol	Typical Description
	GRAVEL; clean to some silt, clay, and sand
	Sandy GRAVEL; clean to some silt and clay
	Silty GRAVEL; up to some clay and sand
	Clayey GRAVEL; up to some silt and sand
	SAND; clean to some silt, clay, and gravel
	Gravelly SAND; clean to some silt and clay
	Silty SAND; up to some clay and gravel
	Clayey SAND; up to some silt and gravel
	SILT; up to some clay, sand, and gravel
	Gravelly SILT; up to some clay and sand
	Sandy SILT; up to some clay and gravel
	Clayey SILT; up to some sand and gravel
	CLAY; up to some silt, sand, and gravel
	Gravelly CLAY; up to some silt and sand
	Sandy CLAY; up to some silt and gravel
	Silty CLAY; up to some sand and gravel
	PEAT
	LANDSCAPE MATERIALS

BEDROCK SYMBOLS

Symbol	Typical Description
	BASALT
	SILTSTONE
	SANDSTONE

SURFACE MATERIAL SYMBOLS

Symbol	Typical Description
	Asphaltic-concrete PAVEMENT
	Portland cement concrete PAVEMENT
	Crushed rock BASE COURSE

SAMPLER SYMBOLS

Symbol	Sampler Description
	2.0-in. O.D. split-spoon sampler and Standard Penetration Test with recovery (ASTM D1586)
	Shelby tube sampler with recovery (ASTM D1587)
	3.0-in. O.D. split-spoon sampler with recovery (ASTM D3550)
	Grab Sample
	Rock core sample interval
	Sonic core sample interval
	Geoprobe sample interval

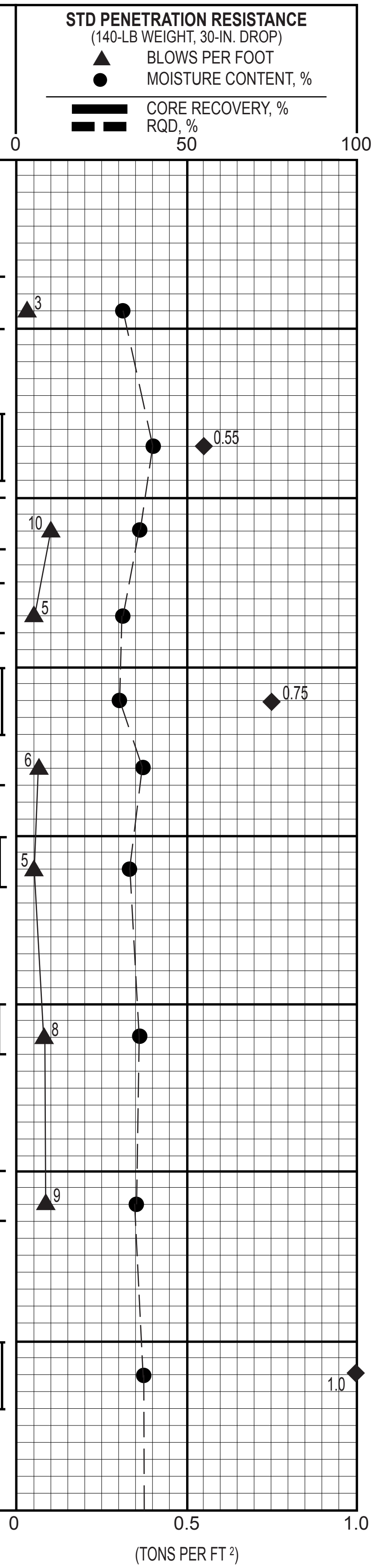
INSTALLATION SYMBOLS

Symbol	Symbol Description
	Flush-mount monument set in concrete
	Concrete, well casing shown where applicable
	Bentonite seal, well casing shown where applicable
	Filter pack, machine-slotted well casing shown where applicable
	Grout, vibrating-wire transducer cable shown where applicable
P	Vibrating-wire pressure transducer
	1-in.-diameter solid PVC
	1-in.-diameter hand-slotted PVC
	Grout, inclinometer casing shown where applicable

FIELD MEASUREMENTS

Symbol	Typical Description
▽	Groundwater level during drilling and date measured
▼	Groundwater level after drilling and date measured
	Rock core recovery
	Rock quality designation (RQD)

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES
		SURFACE ELEVATION 449 ft [±] (NAVD 88)			
		FILL: Crushed Rock	0.9		
		FILL: Soft, brown SILT; trace fine-grained sand, medium plasticity			
5					S-1
		Stiff, brown to dark brown, clayey SILT; some fine- to medium-grained sand, medium plasticity, slight reddish-brown mottling (Landslide Debris)	8.0		S-2
10		----------medium stiff below 11.5 ft			S-3
					S-4
15		---------stiff at 15 ft			S-5
					S-6
20					S-7
25					S-8
30		Stiff, brown and orange-brown, clayey SILT; medium to low plasticity, trace sand- to gravel-size fragments of extremely soft (R0), predominantly decomposed basalt (Landslide Debris)	30.0		S-9
35		----------stiff to very stiff below 35 ft			S-10
40					



2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
NX CORE RUN
SLOTTED PVC PIPE
Water Level (date)

TORVANE SHEAR STRENGTH, TSF
PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-1

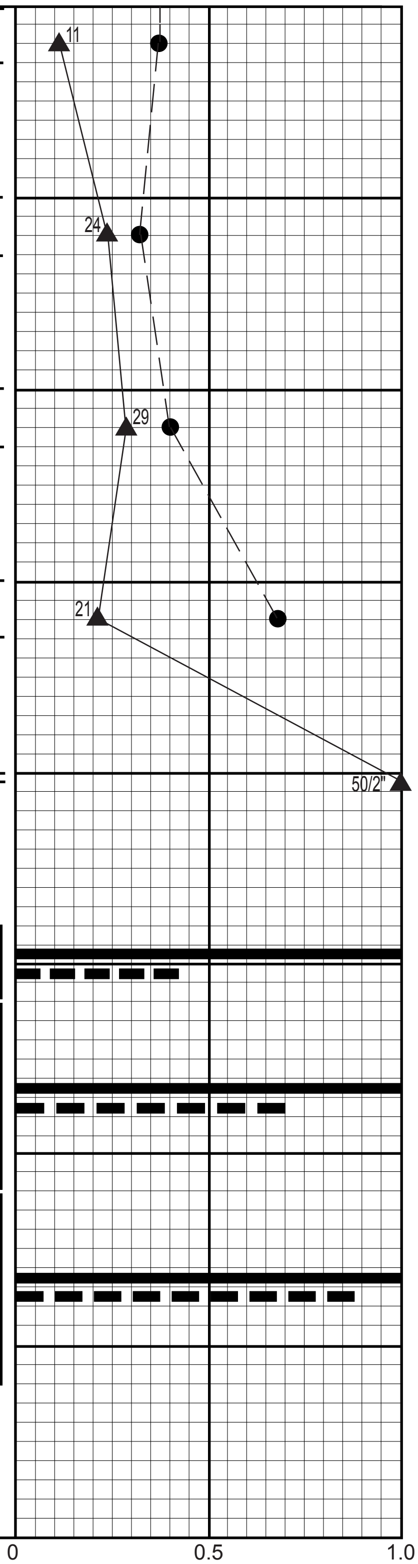
JAN. 2015 JOB. NO. 5338-A FIG. 1A

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)
▲ BLOWS PER FOOT
● MOISTURE CONTENT, %
CORE RECOVERY, %
RQD, %

CLASSIFICATION OF MATERIAL

SURFACE ELEVATION 449 ft [±] (NAVD 88)

DEPTH, FT	CLASSIFICATION OF MATERIAL	DEPTH, FT	SAMPLES
40	Stiff, brown and orange-brown, clayey SILT; medium to low plasticity, trace sand- to gravel-size fragments of extremely soft (R0), predominantly decomposed basalt (Landslide Debris)		S-11
45	Very stiff, red, clayey SILT; trace yellowish-white and black, sand-size fragments of predominantly decomposed basalt, relic rock structure (Landslide Debris)	45.0	S-12
50	Very stiff, gray, clayey SILT; medium to high plasticity, some coarse-grained sand- to fi ne gravel-size fragments of extremely soft (R0), predominantly decomposed basalt (Landslide Debris)	51.0	S-13, S-14
60	Soft to medium hard (R2 to R3), gray and reddish-brown BASALT; coarse-grained sand- to gravel-size fragments of slightly weathered rock (Wanapum Basalt)	60.0	S-15
65	Hard, gray BASALT; vesicular, close to moderately close fractures, fresh to slightly weathered (Wanapum Basalt)	64.0	RUN 1, RUN 2, RUN 3
75	(6/15/2012)	76.0	
80			



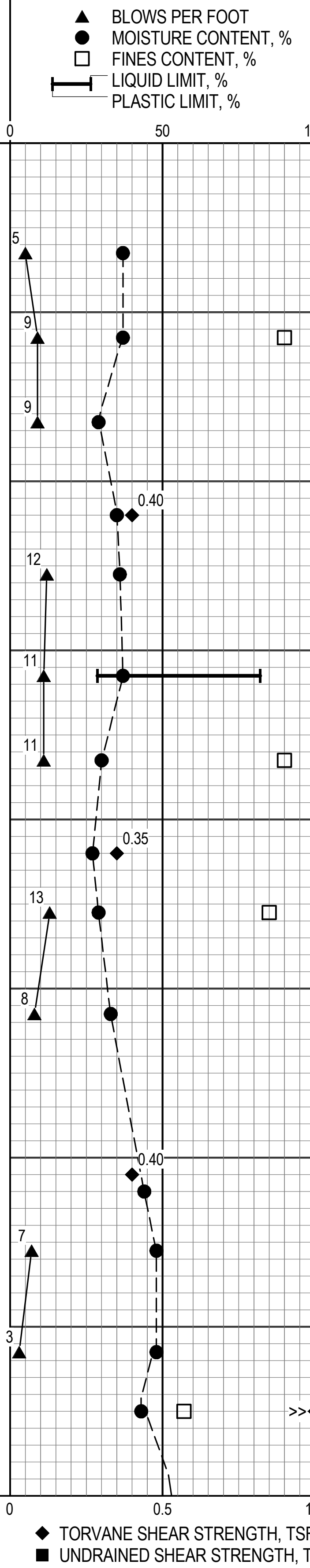
2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
NX CORE RUN
SLOTTED PVC PIPE
Water Level (date)
◆ TORVANE SHEAR STRENGTH, TSF
□ PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-1 (cont.)

JAN. 2015 JOB. NO. 5338-A FIG. 1A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		Surface Elevation: 452.0 ft [±] (NAVD 88)						
0		CRUSHED ROCK (Fill)	451.5 / 0.5					
		SILT, trace to some clay, trace to some fine-grained sand, brown, medium stiff, black manganese oxide staining (Landslide Debris)						
					S-1		2 2 3	
5		---stiff below 5 ft			S-2		3 4 5	
					S-3		3 4 5	
10		---red-brown, some clay to clayey below 10 ft			S-4			Dry Density = 87.8 pcf
					S-5		3 6 6	
15					S-6		3 5 6	
					S-7		5 5 6	
20		---brown to red-brown at 20 ft			S-8			
					S-9		4 6 7	Dry Density = 101.5 pcf
25					S-10		3 4 4	
30		BASALT, gray-brown to red-brown, decomposed, extremely soft (R0), manganese oxide mineralization, relic rock structure, consistency of soft to hard soil (Wanapum Basalt; Landslide Debris)	422.0 / 30.0		S-11			Dry Density = 80 pcf
					S-12		2 3 4	
35					S-13		0 0 3	
					S-14			Dry Density = 76 pcf
40								



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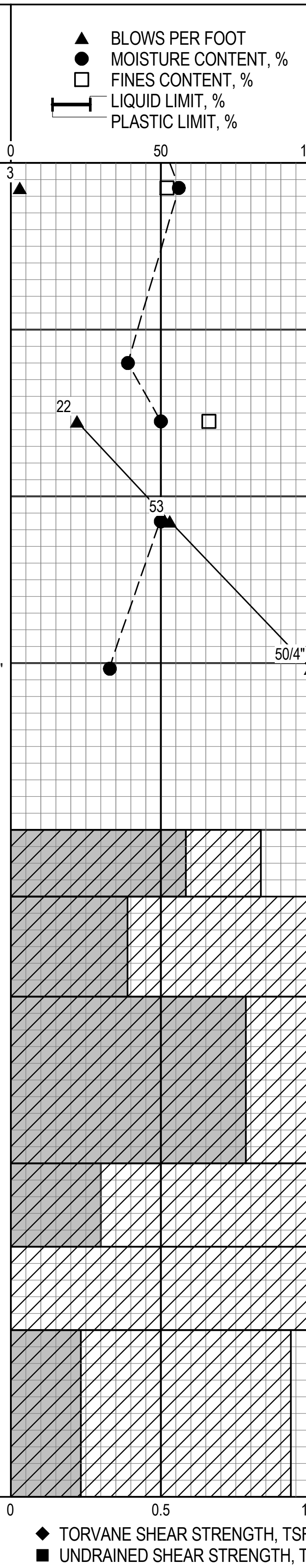
Logged By: M. Marshall	**Drilled by:** Western States Soil Conservation, Inc.	
Date Started: 10/27/14	**Coordinates:** Not Available	
Drilling Method: Mud Rotary **Equipment:** CME 75 HT Truck-Mounted Drill Rig **Hole Diameter:** 3.875 in.		**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 71.7 %
Note: See Legend for Explanation of Symbols		

GRI

BORING B-2

JAN. 2015 JOB NO. 5338-A FIG. 2A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL **Surface Elevation:** 452.0 ft [±] (NAVD 88)	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		BASALT, gray-brown to red-brown, decomposed, extremely soft (R0), manganese oxide mineralization, relic rock structure, consistency of soft to hard soil (Wanapum Basalt; Landslide Debris)			S-15		1 1 2	41.2 ft (1/7/15) 41.6 ft (11/18/14)
45					S-16			Dry Density = 84 pcf
					S-17		6 11 11	
50					S-18		23 23 30	
55		Gravel- to boulder-size fragments of BASALT, gray, decomposed to moderately weathered, soft to medium hard (R2 to R3) (Wanapum Basalt)	397.0 55.0		S-19		50/4"	
60		BASALT, some vesicles, gray, fresh, hard (R4), close joints (Wanapum Basalt)	392.0 60.0		R-1			
					R-2			
65					R-3			
70		---some vesicles below 70 ft			R-4			
					R-5			
75					R-6			
80			373.0 79.0					



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GRI BORING LOG GRI DATA TEMPLATE.GDT 1/16/15

GRI

BORING B-2

JAN. 2015 JOB NO. 5338-A FIG. 2A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL **Surface Elevation:** 452.0 ft [±] (NAVD 88)	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	▲ BLOWS PER FOOT ● MOISTURE CONTENT, % □ FINES CONTENT, % LIQUID LIMIT, % PLASTIC LIMIT, % 0 50 100	**COMMENTS AND ADDITIONAL TESTS**
		BASALT, scoriaceous, dark red to black, slightly to moderately weathered, soft to medium hard (R2 to R3), 6-in.-thick zone with black amorphous glassy luster (carbonized wood) (Vantage Horizon of the Grande Ronde Basalt)			R-7				
85		BASALT, some vesicles, dark gray, moderately weathered, soft (R2), very close fractures, some healed (Grande Ronde Basalt)	367.0 85.0		R-8				
		---highly vesicular below 89 ft							
90		(10/28/2014)	362.0 90.0						
95									
100									
105									
110									
115									
120									

0 0.5 1.0

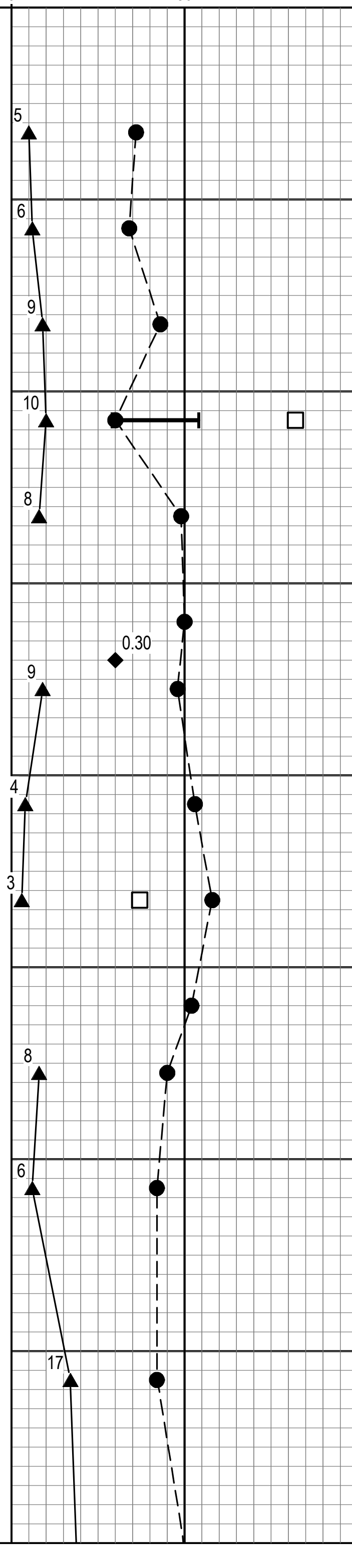
◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF

GRI

BORING B-2

JAN. 2015 JOB NO. 5338-A FIG. 2A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	▲ BLOWS PER FOOT ● MOISTURE CONTENT, % □ FINES CONTENT, % LIQUID LIMIT, % PLASTIC LIMIT, % (0 50 100)	COMMENTS AND ADDITIONAL TESTS
		Surface Elevation: 446.0 ft [±] (NAVD 88)							
		CRUSHED ROCK (Fill)	445.5 / 0.5						
		SILT, trace to some clay, trace to some fine-grained sand, brown, medium stiff (Fill)			S-1		1 2 3	5	
5		SILT, some clay to clayey, trace to some fine-grained sand, brown, medium stiff (Landslide Debris)	441.0 / 5.0		S-2		0 3 3	6	
					S-3		2 4 5	9	
10					S-4		3 5 5	10	
		BASALT, gray-brown to red-brown, decomposed, extremely soft (R0), secondary mineralization, relict rock structure, consistency of soft to hard soil (Wanapum Basalt, Landslide Debris)	433.5 / 12.5		S-5		3 5 3	8	
15					S-6			0.30	Dry Density = 76 pcf
					S-7		3 4 5	9	
20					S-8		3 3 1	4	
					S-9		2 1 2	3	22.7 ft (11/18/14; 1/7/15)
25					S-10				Dry Density = 68 pcf
					S-11		3 4 4	8	
30					S-12		3 3 3	6	
		---boulder from 33 to 34.5 ft							
35		---gravel-size fragments of moderately weathered, soft (R2) basalt below 35 ft			S-13		5 8 9	17	
40									



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◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF

Logged By: M. Marshall	**Drilled by:** Western States Soil Conservation, Inc.	
Date Started: 10/28/14	**Coordinates:** Not Available	
Drilling Method: Mud Rotary **Equipment:** CME 75 HT Truck-Mounted Drill Rig **Hole Diameter:** 3.875 in.		**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 71.7 %
Note: See Legend for Explanation of Symbols		

GRI

BORING B-3

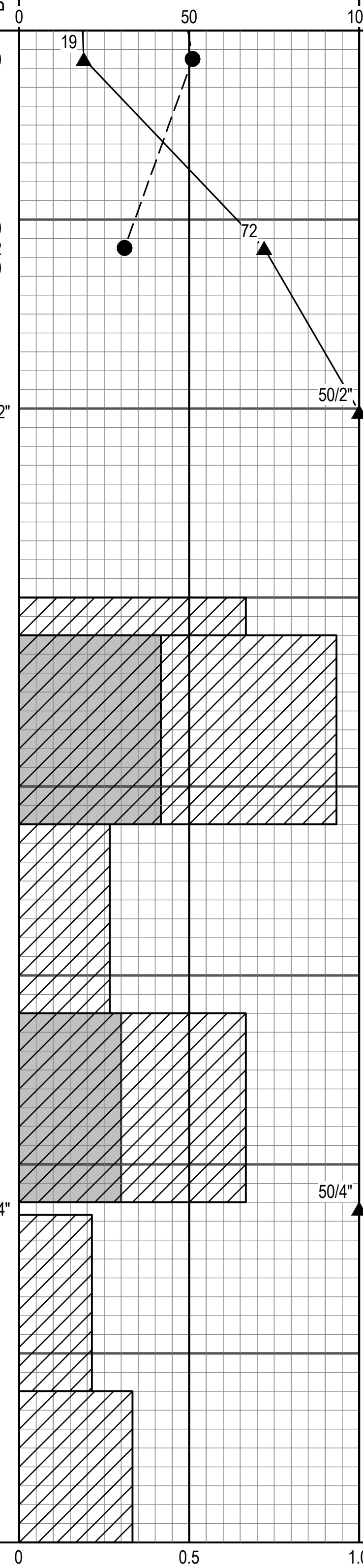
JAN. 2015 JOB NO. 5338-A FIG. 3A

CLASSIFICATION OF MATERIAL

Surface Elevation: 446.0 ft [±] (NAVD 88)

▲ BLOWS PER FOOT
● MOISTURE CONTENT, %
□ FINES CONTENT, %
LIQUID LIMIT, %
PLASTIC LIMIT, %

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	SAMPLE NO.	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
	BASALT, light brown, decomposed to moderately weathered, extremely soft to soft (R0 to R2), secondary mineralization, relict rock structure, consistency of soft to hard soil (Wanapum Basalt, Landslide Debris)		S-14	4 10 9	
45			S-15	20 32 40	
			S-16	50/2"	
50	Gravel- to boulder-size fragments of BASALT, gray, slightly to moderately weathered, medium hard to hard (R3 to R4) (Wanapum Basalt)	396.0 / 50.0			
55	BASALT, gray, slightly weathered, medium hard to hard (R3 to R4), close joints, black carbonized wood within near-vertical (80°) closed fractures, some with chilled margin (Wanapum Basalt)	391.0 / 55.0	R-1		
			R-2		
60			R-3		
65	---some vesicles, fresh to slightly weathered, hard (R4), several closed near-vertical fractures, iron and manganese oxide staining along joints below 65 ft		R-4		
70					
	BASALT, highly vesicular, red-brown, moderately weathered, soft to very soft (R2 to R1), secondary mineralization (Vantage Horizon of the Grande Ronde Basalt)	375.0 / 71.0	S-17	50/4"	
			R-5		
75			R-6		
80					



(CONTINUED NEXT PAGE)

◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF

GRI BORING LOG GRI DATA TEMPLATE.GDT 1/16/15

GRI

BORING B-3

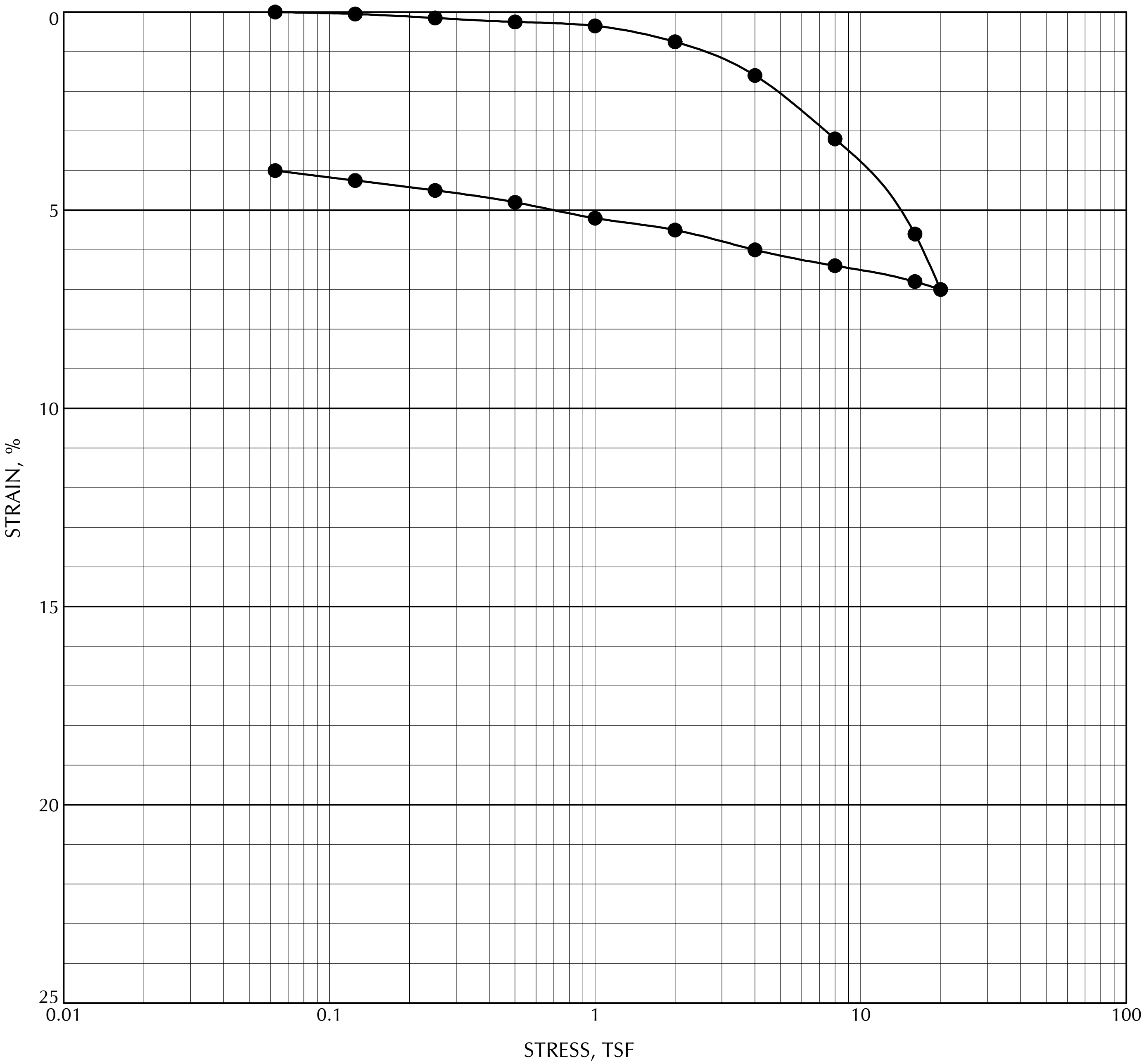
DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL **Surface Elevation:** 446.0 ft [±] (NAVD 88)	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	▲ BLOWS PER FOOT ● MOISTURE CONTENT, % □ FINES CONTENT, % LIQUID LIMIT, % PLASTIC LIMIT, % (0 – 50 – 100)	COMMENTS AND ADDITIONAL TESTS
		BASALT, highly vesicular, red-brown, moderately weathered, soft to very soft (R2 to R1), secondary mineralization (Vantage Horizon of the Grande Ronde Basalt) (10/29/2014)	365.0 / 81.0						
85									
90									
95									
100									
105									
110									
115									
120									

0 0.5 1.0

◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF

GRI

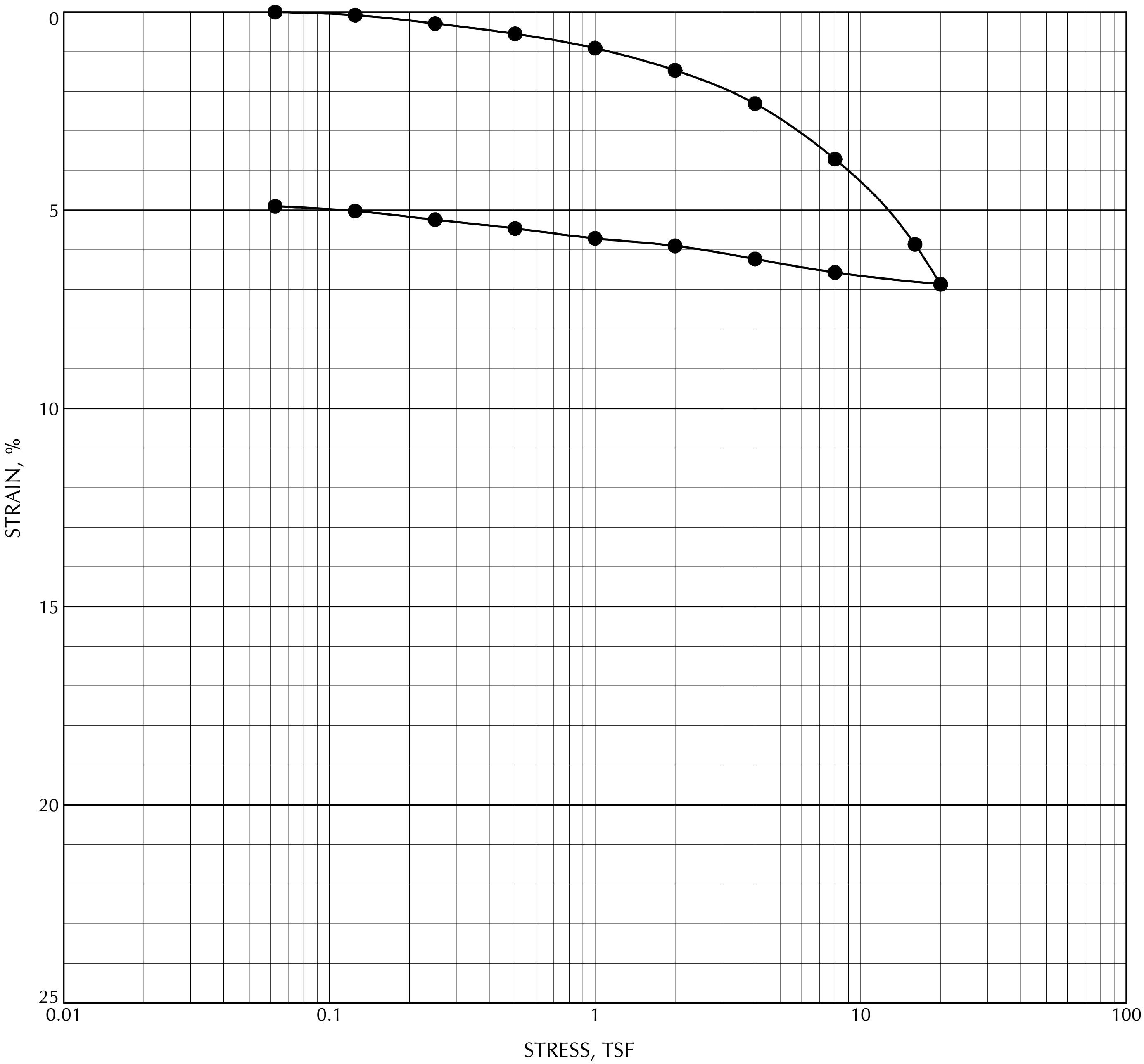
BORING B-3



	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	Initial MC, %
●	B-1	S-5	16.5	Clayey SILT, some fine- to medium-grained sand, brown, medium stiff (Landslide Debris)	89	33

GRI

CONSOLIDATION TEST



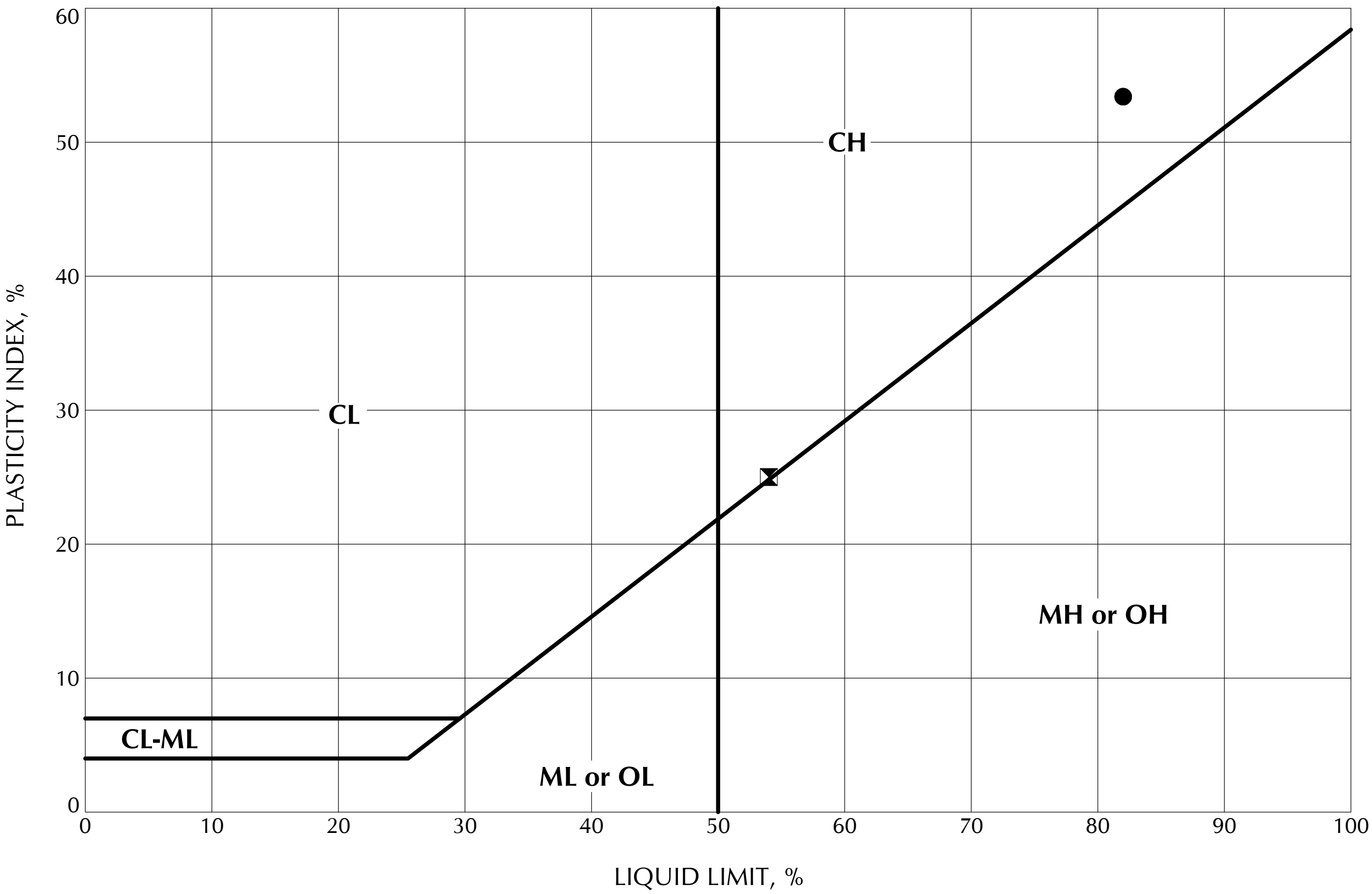
	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	Initial MC, %
●	B-2	S-14	37.3	BASALT, gray-brown to red-brown, decomposed, extremely soft (R0), manganese oxide mineralization, relic rock structure, consistency of soft to hard soil (Wanapum Basalt; Landslide Debris)	80	43

GRI

CONSOLIDATION TEST

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
ML	INORGANIC CLAYEY SILTS TO VERY FINE SANDS OF SLIGHT PLASTICITY
CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
MH	INORGANIC SILTS AND CLAYEY SILT
CH	INORGANIC CLAYS OF HIGH PLASTICITY



	Location	Sample	Depth, ft	Classification	LL	PL	PI	MC, %
●	B-2	S-6	15.0	SILT, some clay to clayey, trace to some fine-grained sand, red-brown (Landslide Debris)	82	29	53	37
⧗	B-3	S-4	10.0	SILT, some clay to clayey, trace to some fine-grained sand, brown (Landslide Debris)	54	29	25	30

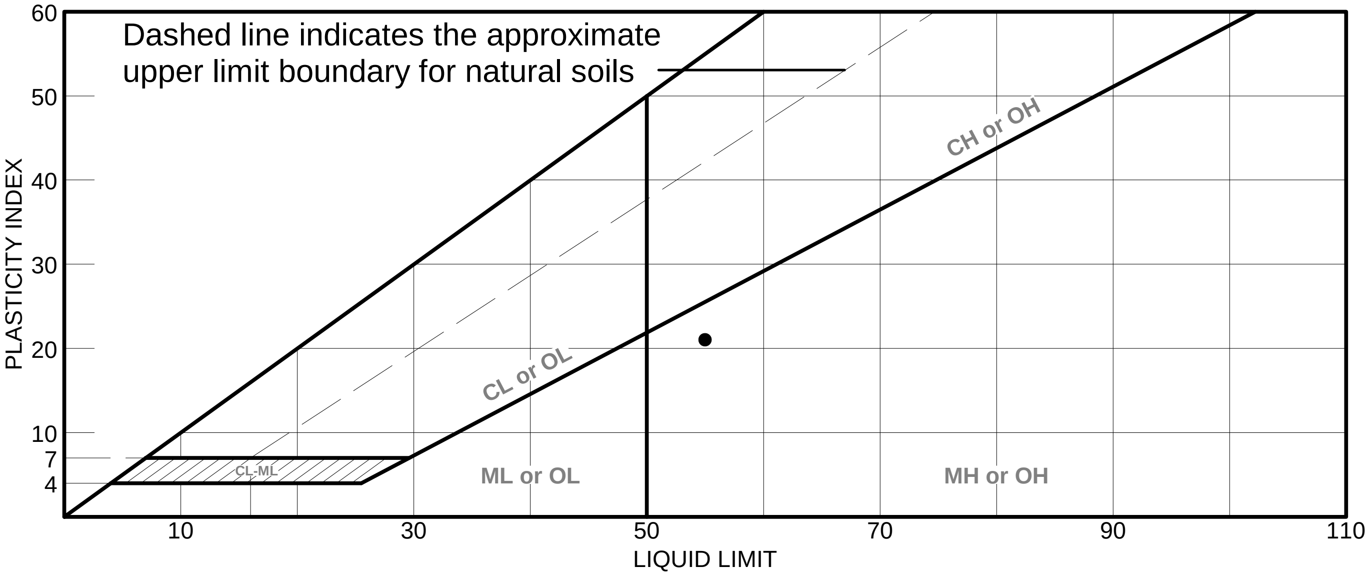
ATTERBERG-PLASTICITY 2 PER PAGE GRI DATA TEMPLATE.GDT 1/16/15

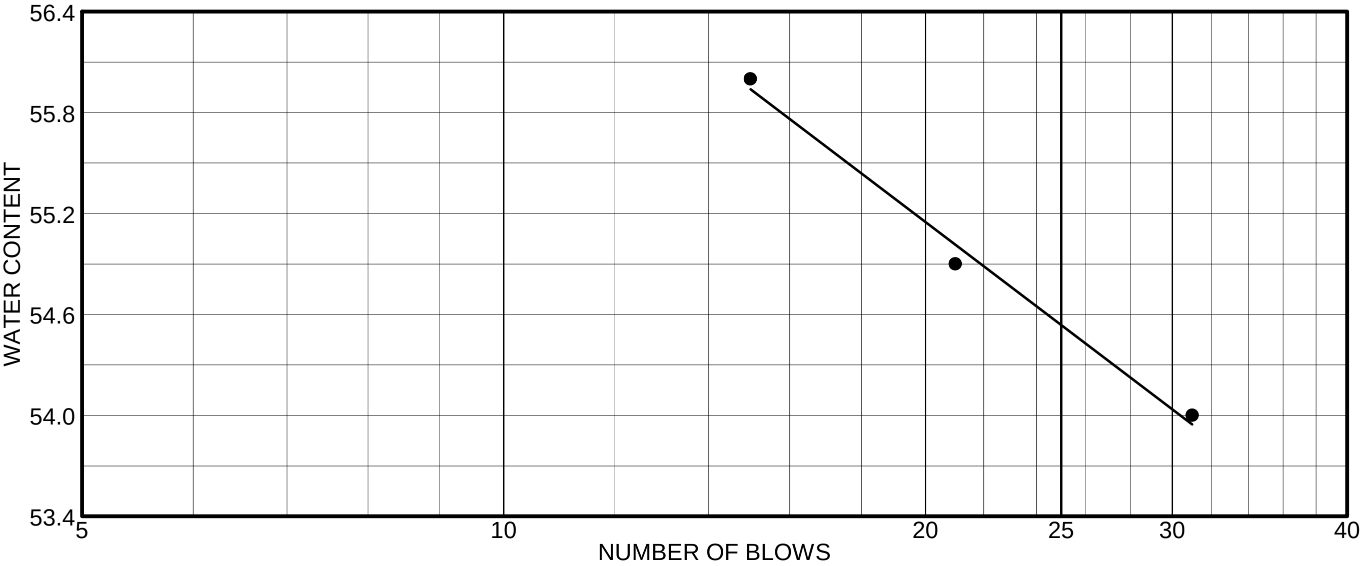
GRI

PLASTICITY CHART

JAN. 2015 JOB NO. 5338-A FIG. 6A

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Strong Brown Sandy Elastic SILT	55	34	21	87.2	60.1	MH

Project No. 823-005 **Client:** GRI

Project: Bolton Reservoir - 5338-A

● **Source:** B-2 **Sample No.:** S-13 **Elev./Depth:** 35'

Remarks:

●

LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Figure

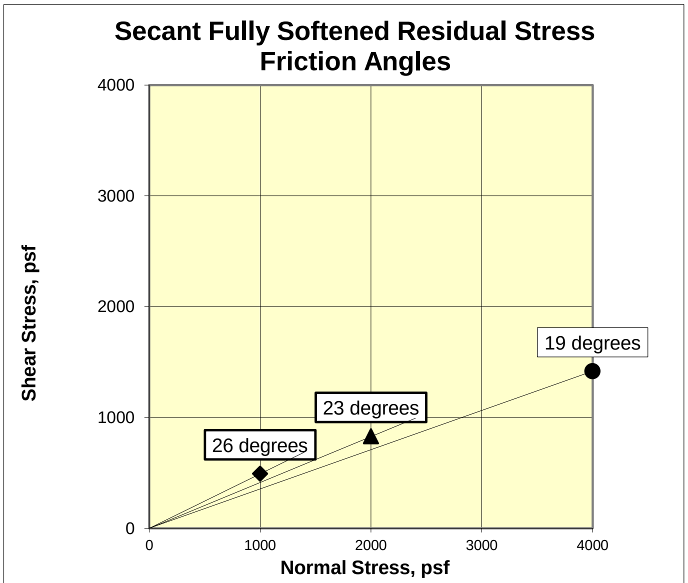
Drained Residual Torsional Shear Strength (ASTM D6467)

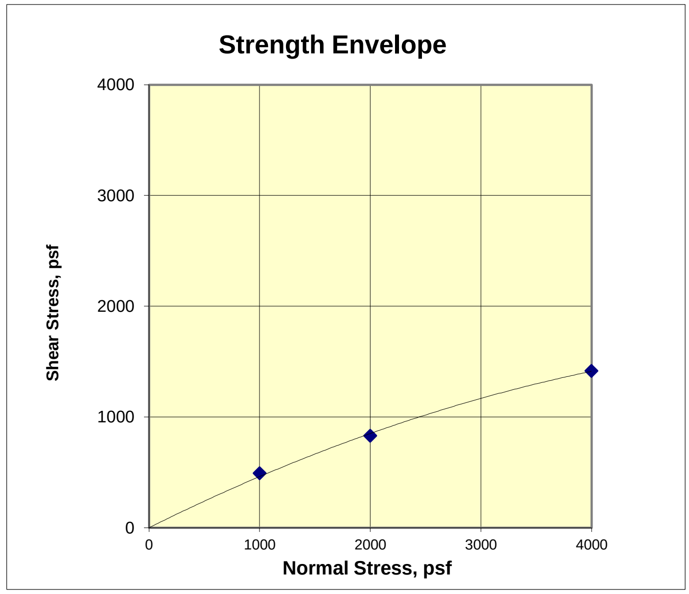
CTL Job No.:	823-005	**Boring:**	B-2	**Date:**	12/9/2014	**Clay, %:**	29.2
Client:	GRI	**Sample:**	S-13	**By:**	PJ	**LL:**	55
Project Name:	Bolton Reservoir	**Depth (ft):**	35	**Checked:**	DC	**PL:**	34
Project Number:	5338-A	**Test Type:**	Fully Softened Residual				

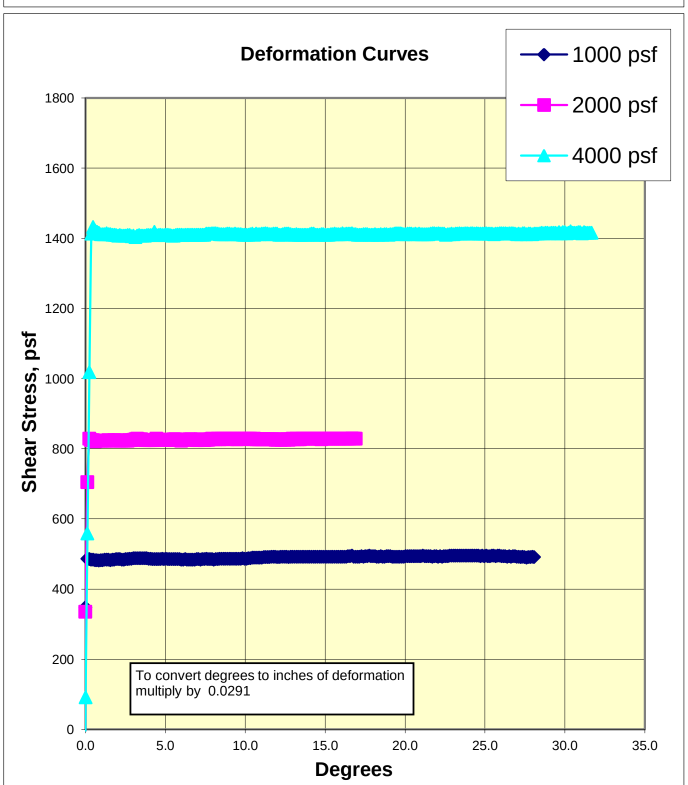
Soil Type: Strong Brown Sandy Elastic SILT

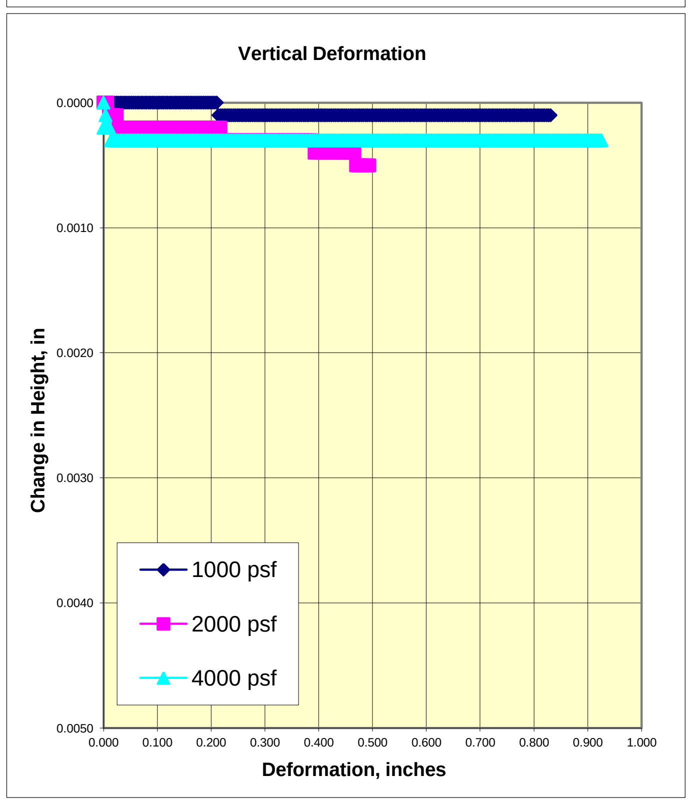
Normal Stress, psf:	1000	2000	4000
Secant Phi, deg.:	26	23	19

Remarks: This sample has an unusually highly curved strength envelope.

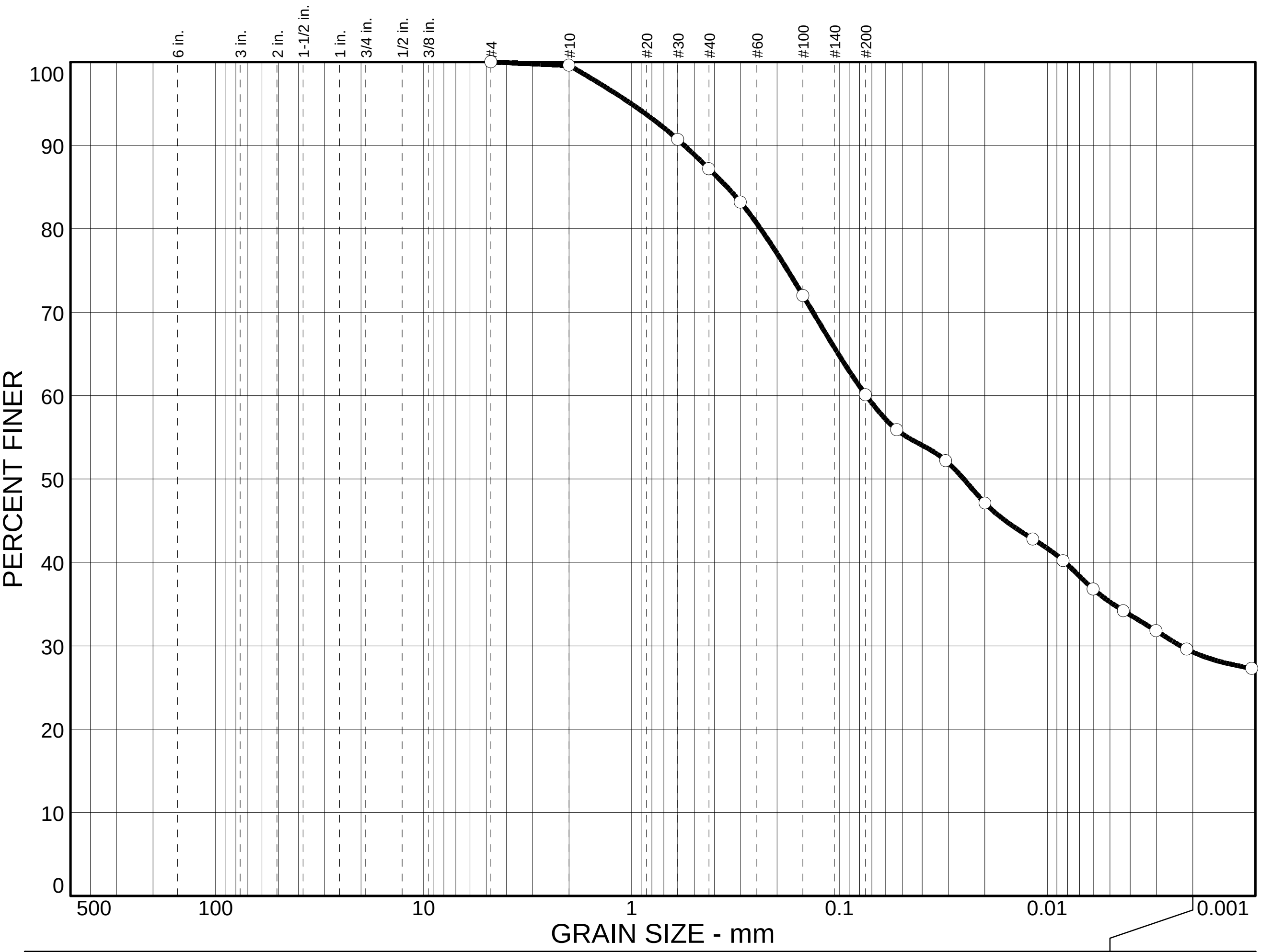








Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	39.9	30.9	29.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.6		
#30	90.7		
#40	87.2		
#50	83.2		
#100	72.0		
#200	60.1		
#270	55.9		
0.0308 mm.	52.2		
0.0200 mm.	47.1		
0.0118 mm.	42.8		
0.0084 mm.	40.2		
0.0060 mm.	36.8		
0.0043 mm.	34.2		
0.0030 mm.	31.8		
0.0021 mm.	29.6		
0.0010 mm.	27.3		

* (no specification provided)

Soil Description

Strong Brown Sandy Elastic SILT

Atterberg Limits

PL= 34 LL= 55 PI= 21

Coefficients

D_{85}= 0.348 D_{60}= 0.0745 D_{50}= 0.0253
D_{30}= 0.0023 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= MH AASHTO=

Remarks

Sample No.: S-13 **Source of Sample:** B-2 **Date:** 12/3/14
Location: **Elev./Depth:** 35'

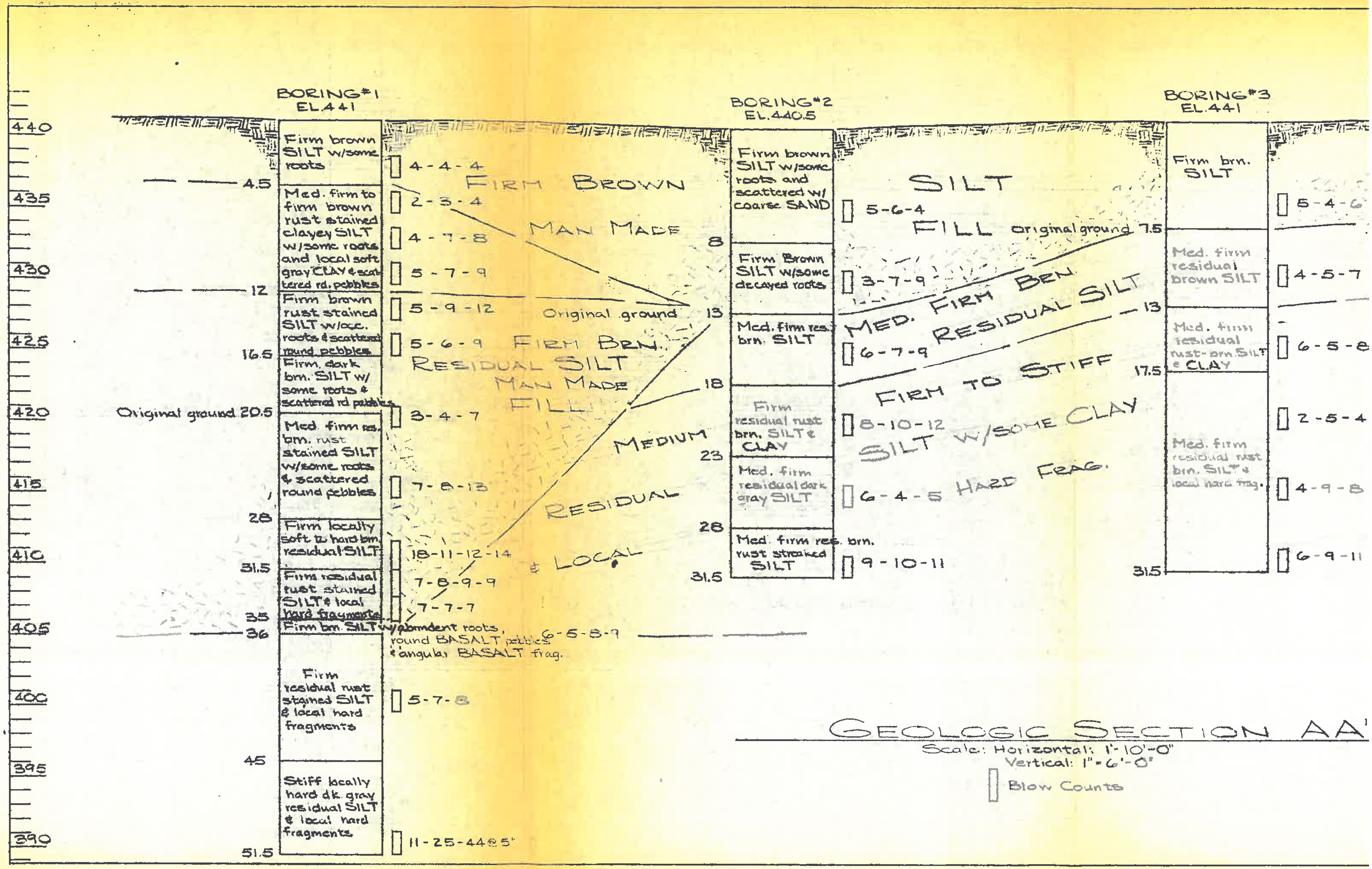
COOPER TESTING LABORATORY

Client: GRI
Project: Bolton Reservoir - 5338-A

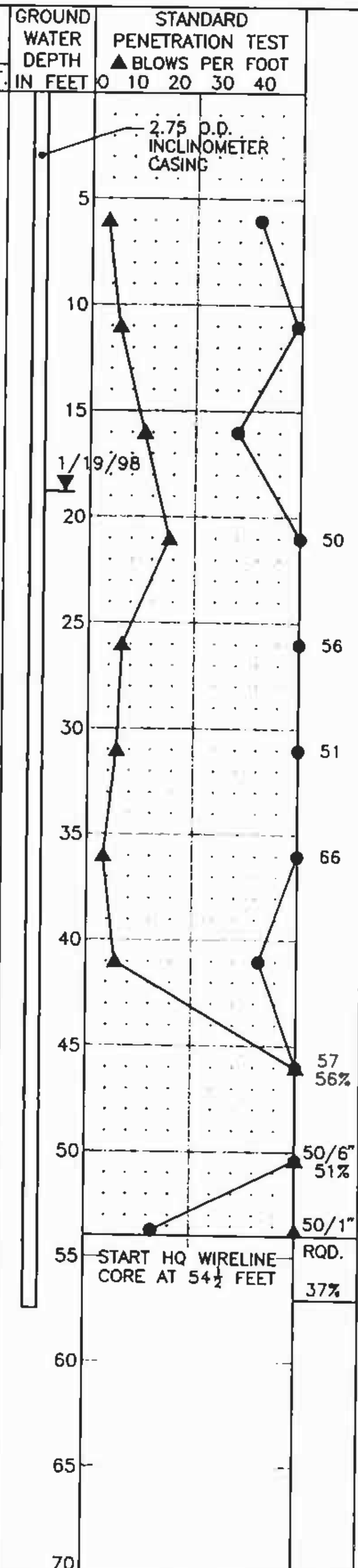
Project No: 823-005 **Figure**

APPENDIX B

Logs of Subsurface Explorations by Others


BORING#1
EL.441
BORING#2
EL.440.5
BORING#3
EL.441
440
435
430
425
420
415
410
405
400
395
390
Firm brown SILT w/some roots
4.5
Med. firm to firm brown rust stained clayey SILT w/some roots and local soft gray CLAY & scattered rd. pebbles
12
Firm brown rust stained SILT w/occ. roots & scattered round pebbles
16.5
Firm, dark brn. SILT w/ some roots & scattered rd pebbles
Original ground 20.5
Med. firm res. brn. rust stained SILT w/some roots & scattered round pebbles
28
Firm locally soft to hard brn. residual SILT
31.5
Firm residual rust stained SILT & local hard fragments
35
36
Firm brn. SILT w/abundent roots, round BASALT pebbles & angular BASALT frag.
Firm residual rust stained SILT & local hard fragments
45
Stiff locally hard dk. gray residual SILT & local hard fragments
51.5
4 - 4 - 4
2 - 3 - 4
4 - 7 - 8
5 - 7 - 9
5 - 9 - 12
5 - 6 - 9
3 - 4 - 7
7 - 8 - 13
18 - 11 - 12 - 14
7 - 8 - 9 - 9
7 - 7 - 7
6 - 5 - 8 - 9
5 - 7 - 8
11 - 25 - 44@5"
FIRM BROWN
MAN MADE
Original ground
FIRM BRN.
RESIDUAL SILT
MAN MADE
FILL
MEDIUM
RESIDUAL
& LOCAL
Firm brown SILT w/some roots and scattered w/ coarse SAND
8
Firm Brown SILT w/some decayed roots
13
Med. firm res. brn. SILT
18
Firm residual rust brn. SILT & CLAY
23
Med. firm residual dark gray SILT
28
Med. firm res. brn. rust streaked SILT
31.5
5-6-4
3-7-9
6-7-9
8-10-12
6-4-5
9-10-11
SILT
FILL
original ground 7.5
MED. FIRM BRN.
RESIDUAL SILT
13
FIRM TO STIFF
SILT W/SOME CLAY
HARD FRAG.
Firm brn. SILT
Med. firm residual brown SILT
Med. firm residual rust-brn. SILT & CLAY
17.5
Med. firm residual rust brn. SILT & local hard frag.
31.5
5 - 4 - 6
4 - 5 - 7
6 - 5 - 8
2 - 5 - 4
4 - 9 - 8
6 - 9 - 11
GEOLOGIC SECTION AA'
Scale: Horizontal: 1"-10'-0"
Vertical: 1"=6'-0"
Blow Counts

ELEV.	DEPTH IN FEET	MATERIAL DESCRIPTION	SAMPLES NO.	S.P.T.
		SURFACE ELEV. (FT.) 437 (approx.)		
		SOFT, to MEDIUM STIFF, brown, slightly clayey to clayey SILT (FILL).	1	4
			2	7
424	13	SOFT to STIFF, mottled orange-brown and gray, sandy, slightly clayey SILT (DECOMPOSED BASALT).	3	13
			4	19
			5	8
		...becomes gravelly	6	7
		...becomes gray	7	4
			8	7
395	42	VERY DENSE, gray, slightly clayey, sandy SILT; relic rock texture (DECOMPOSED BASALT).	9	57
			10	50/6"
			11	50/1"
382½	54½	SOFT to MEDIUM HARD, grey, moderately weatherd BASALT; highly jointed.	RUN NO. 1	CORE RECV. 90%
379½	57½	Bottom of Boring: 57½ feet		



LEGEND

- 2" S.P.T. SAMPLE
- * SAMPLE NOT RECOVERED
- U 3" THIN WALL SAMPLE
- P PITCHER SAMPLE
- IMPERVIOUS SEAL
- WATER LEVEL
- PIEZOMETER TIP
- LIQUID LIMIT / NATURAL WATER CONTENT / PLASTIC LIMIT
- • WATER CONTENT IN PERCENT

NOTES

1. MATERIAL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.
2. WATER LEVEL IS FOR DATE SHOWN AND MAY VARY WITH TIME OF YEAR.
3. THE ANNULUS AROUND THE INCLINOMETER CASING WAS BACKFILLED USING A TREMIED CEMENT-BENTONITE GROUT.
4. WATER LEVEL MEASURED IN ADJACENT PIEZOMETER DRILL HOLE LT-1P.

C1057\BORLOG01

DRILLER Geo-Tech Explorations
DATE START 12/5/97 FINISH 12/5/97
DRILLING TECHNIQUE 4⅞-inch O.D. Tricone Mud Rotary, HSA 8-inch O.D., HQ Coring

Landslide Technology
10250 S.W. Greenburg Rd. Portland, OR 97223

TITLE SUMMARY BORING LOG LT-1

JOB BOLTON RESERVOIR LANDSLIDE WEST LINN, OREGON

DATE JAN 1998

JOB NO. 1057

FIG. 2

APPENDIX C

Site-Specific Seismic Hazard Study

APPENDIX C

SITE-SPECIFIC SEISMIC HAZARD STUDY

General

GRI has completed a site-specific seismic hazard study for the proposed Bolton Reservoir in West Linn, Oregon. The purpose of the study was to evaluate potential seismic hazards associated with regional and local seismicity. The site-specific hazard study is intended to meet the requirements of the 2012 International Building Code (IBC), which was recently adopted by the 2014 Oregon Structural Specialty Code (OSSC). The 2012 IBC is based on the American Society of Civil Engineers (ASCE) 7-10 document *Minimum Design Loads for Buildings and Other Structures*. Our work was based on the potential for regional and local seismic activity, as described in the existing scientific literature, and on the subsurface conditions at the site, as disclosed by the subsurface explorations completed for this project. Specifically, our work included the following tasks:

1) A detailed review of available literature, including published papers, maps, open-file reports, seismic histories and catalogs, , and other sources of available information regarding the tectonic setting, regional and local geology, and historical seismic activity that might have a significant effect on the site.

2) Compilation and evaluation of subsurface data collected at and in the vicinity of the site, including classification and laboratory analysis of soil samples. This information was used to prepare a generalized subsurface profile for the site.

3) Identification of the potential seismic events (earthquakes) appropriate for the site and characterization of those events in terms of a generalized design event.

4) Office studies, based on the generalized subsurface profile and the generalized design earthquake, resulting in conclusions and recommendations concerning:

   a) specific seismic events that might have a significant effect on the site,

   b) the potential for seismic energy amplification and liquefaction or soil strength loss at the site, and

   c) site-specific acceleration response spectra for design of the proposed reservoir.

This appendix describes the work accomplished and summarizes our conclusions and recommendations.

Geologic Setting

On a regional scale, the site is located at the northern end of the Willamette Valley, a broad, gently deformed, north-south-trending topographic feature separating the Coast Range to the west from the Cascade Mountains to the east. The site is located approximately 100 km inland from the Cascadia Subduction Zone (CSZ), an active plate boundary along which remnants of the Farallon plate (the Gorda, Juan de Fuca, and Explorer plates) are being subducted beneath the western edge of the North American plate. The subduction zone is a broad, eastward-dipping zone of contact between the upper portion of the subducting slabs of the Gorda, Juan de Fuca, and Explorer plates and the over-riding North American plate as shown on the Tectonic Setting Summary, Figure 1C.

On a local scale, the site is located in the Portland Basin, a large, well-defined, northwest-trending structural basin bounded by high-angle, northwest-trending, right-lateral strike-slip faults considered to be seismogenic. The distribution of these faults relative to the site is shown on the Regional Geologic Map, Figure 2C. Additional faults in the project area that are considered potentially active by the U. S. Geological Survey (USGS) are shown on the Local Fault Map, Figure 3C. Information regarding the continuity and potential activity of these faults is lacking, due largely to the scale at which geologic mapping in the area has been conducted and the presence of thick, relatively young, basin-filling sediments that obscure underlying structural features. Other faults may be present within the basin, but clear stratigraphic and/or geophysical evidence regarding their location and extent is not presently available. Additional discussion regarding crustal faults is provided in the Local Crustal Event section below.

Because of the proximity of the site to the CSZ and its location within the Portland Basin, three distinctly different sources of seismic activity contribute to the potential for the occurrence of damaging earthquakes. Each of these sources is generally considered to be capable of producing damaging earthquakes. Two of these sources are associated with the deep-seated tectonic activity related to the subduction zone; the third is associated with movement on the local, relatively shallow structures within and adjacent to the Portland Basin.

The site is located on the eastern flank of the Tualatin Mountains, a topographic upland that separates the Portland Basin to the northeast from the Tualatin Basin to the west and the Willamette Valley to the south. Geologic mapping completed for the area indicates the site is located in the vicinity of the contact between the Miocene-age Wanapum Basalt and the Grande Ronde Basalt units of the Columbia River Basalt Group (Madin, 2009). The site and other areas of the Tualatin Mountain upland are capped by deposits of fine-grained, wind-blown silt, referred to as Portland Hills Silt. Quaternary alluvial deposits associated with the Willamette River and the Ice Age Missoula Floods (about 15,000 to 20,000 years ago) are present northeast of the site, north of Hwy 43.

Seismicity

General. The geologic and seismologic information available for identifying the potential seismicity at the site is incomplete, and large uncertainties are associated with estimates of the probable magnitude, location, and frequency of occurrence of earthquakes that might affect the site. The available information indicates the potential seismic sources that may affect the site can be grouped into three independent categories: *subduction zone events* related to sudden slip between the upper surface of the Juan de Fuca plate and the lower surface of the North American plate, *subcrustal events* related to deformation and volume changes within the subducted mass of the Juan de Fuca plate, and *local crustal events* associated with movement on shallow, local faults within and adjacent to the Portland Basin. Based on our review of currently available information, we have developed generalized design earthquakes for each of these categories in accordance with Section 1803 of the OSSC. The design earthquakes are characterized by three important properties: size, location relative to the subject site, and the peak horizontal bedrock accelerations produced by the event. In this study, earthquake size is expressed by the moment magnitude

(M); location is expressed as the closest distance to the fault rupture, measured in kilometers; and peak horizontal bedrock accelerations are expressed in units of gravity (1 g = 32.2 ft/sec^2 = 981 cm/sec^2).

Subduction Zone Event. The last interplate earthquake on the CSZ occurred in January 1700. Geological studies show that great megathrust earthquakes have occurred repeatedly in the past 7,000 years (Atwater et al., 1995; Clague, 1997; Goldfinger, 2003; and Kelsey et al., 2005), and geodetic studies (Hyndman and Wang, 1995; Savage et al., 2000) indicate rate of strain accumulation consistent with the assumption that the CSZ is locked beneath offshore northern California, Oregon, Washington, and southern British Columbia (Fluck et al., 1997; Wang et al., 2001). Numerous geological and geophysical studies suggest the CSZ may be segmented (Hughes and Carr, 1980; Weaver and Michaelson, 1985; Guffanti and Weaver, 1988; Goldfinger, 1994; Kelsey et al., 1994; Mitchell et al., 1994; Personius, 1995; Nelson and Personius, 1996; Witter, 1999), but the most recent studies suggest that for the last great earthquake in 1700, most of the subduction zone ruptured in a single M9.0 earthquake (Satake et al., 1996; Atwater and Hemphill-Haley, 1997; Clague et al., 2000). Published estimates of the probable maximum size of subduction zone events range from M8.3 to greater than M9.0. Numerous detailed studies of coastal subsidence, tsunamis, and turbidites yield a wide range of recurrence intervals, but the most complete records (>4,000 years) indicate average intervals of 350 to 600 years between great earthquakes on the CSZ (Adams, 1990; Atwater and Hemphill-Haley, 1997; Witter, 1999; Clague et al., 2000; Kelsey et al., 2002; Kelsey et al., 2005; Witter et al., 2003; Goldfinger et al, 2012). Tsunami inundation in buried marshes along the Washington and Oregon coast and stratigraphic evidence from the Cascadia margin support these recurrence intervals (Kelsey et al., 2005; Goldfinger, 2003).

The USGS probabilistic analysis assumes four potential locations for the location of the eastern edge of the earthquake rupture zone as shown on Figure 4C. The 2008 USGS mapping effort indicates two rupture scenarios are assumed to represent these megathrust events: 1) M9±0.2 events that rupture the entire CSZ every 500 years and 2) M8.3 to 8.7 events with rupture zones that occur on segments of the CSZ and occur over the entire length of the CSZ during a period of about 500 years (Petersen et al., 2008). The assumed distribution of earthquakes is shown on the Assumed Magnitude-Frequency Distribution, Figure 5C. This distribution assumes the larger M9.0 earthquake is the most likely single CSZ earthquake scenario, as also indicated by the USGS deaggregation for the site. Therefore, for our deterministic analysis, we have chosen to represent the subduction zone event by a design earthquake of M9.0 at a focal depth of 20 km and rupture distance of 100 km. This corresponds to a sudden rupture of the whole length of the Juan de Fuca-North American plate interface with an assumed rupture zone due west of the site. Based on an average of the attenuation relationships published by Youngs et al. (1997), Atkinson and Boore (2003), and Zhao et al. (2006), a subduction zone earthquake of this size and location would result in a peak horizontal bedrock acceleration of approximately 0.12 g at the site.

Subcrustal Event. There is no historic earthquake record of subcrustal, intraslab earthquakes in Oregon. Although both the Puget Sound and northern California region have experienced many of these earthquakes in historic times, Wong (2005) hypothesizes that due to subduction zone geometry, geophysical conditions, and local geology, Oregon may not be subject to intraslab earthquakes. In the Puget Sound area, these moderate to large earthquakes are deep (40 to 60 km) and over 200 km from the deformation front of the subduction zone. Offshore, along the northern California coast, the earthquakes are shallower (up to 40 km) and located along the deformation front. Estimates of the probable size, location, and frequency of subcrustal events in Oregon are generally based on comparisons of the CSZ with active convergent plate margins in other parts of the world and on the historical seismic record for the region surrounding Puget Sound, where significant events known to have occurred within the subducting Juan de Fuca plate have been recorded. Published estimates of the probable maximum size of these events range from M7.0 to 7.5. The 1949, 1965, and 2001 documented subcrustal earthquakes in the Puget Sound area correspond to M7.1, 6.5, and 6.8, respectively. Published information regarding the location and geometry of the subducting zone indicates that a focal depth of 50 km is probable (Weaver and Shedlock, 1989). We have chosen to represent the subcrustal event by a design earthquake of M7.0 at a focal depth of 50 km and a rupture distance of 60 km. Based on the attenuation relationships published by Youngs et al. (1997) and Atkinson and Boore (2003), a subcrustal earthquake of this size and location would result in a peak horizontal bedrock acceleration of approximately 0.14 g at the site.

Local Crustal Event. Sudden crustal movements along relatively shallow, local faults in the project area, although rare, have been responsible for local crustal earthquakes. The precise relationship between specific earthquakes and individual faults is not well understood, since few of the faults in the area are expressed at the ground surface, and the foci of the observed earthquakes have not been located with precision. The history of local seismic activity is commonly used as a basis for determining the size and frequency to be expected of local crustal events. Although the historical record of local earthquakes is relatively short (the earliest reported seismic event in the area occurred in 1920), it can serve as a guide for estimating the potential for seismic activity in the area.

Based on fault mapping conducted by the USGS, the Bolton Fault is the closest mapped crustal fault identified as a hazard to the site (USGS, 2008). The surface trace of the Bolton Fault is located about 900 ft northeast of the site (Madin, 2009). The Bolton Fault has a characteristic earthquake magnitude of 6.2. A crustal earthquake of this size and location would result in a peak horizontal bedrock acceleration of approximately 0.45 g at the site based on an average of the NGA ground motion relations published by Boore and Atkinson (2008), Campbell and Bozorgnia (2008), and Chiou and Youngs (2008).

Summary of Deterministic Earthquake Parameters

In summary, three distinctly different types of earthquakes affect seismicity in the project area. Deterministic evaluation of the earthquake sources using recently published attenuation ground motion relations provides estimates of ground response for each individual earthquake type. Unlike probabilistic estimates, these deterministic estimates are not associated with a relative hazard level or probability of occurrence and simply provide an estimate of the ground motion parameters for each type of fault at a given distance from the site. For each earthquake source, we have attempted to use attenuation relationships and weighting that are consistent with the development of the 2008 USGS seismic hazard maps. The basic parameters of each type of earthquake are as follows:

Earthquake Source	Attenuation Relationships for Target Spectra	Magnitude, M	Rupture Distance, km	Focal Depth, km	Peak Bedrock Acceleration, g	Average Peak Bedrock Acceleration, g
Subduction Zone	Youngs et al., 1997	9.0	100	20	0.14	0.12
	Atkinson and Boore, 2003	9.0	100	20	0.07	
	Zhao et al., 2006 (1)	9.0	100	20	0.14	
Subcrustal	Youngs et al., 1997	7.0	60	50	0.15	0.14
	Atkinson and Boore, 2003	7.0	60	50	0.13	

Earthquake Source	Attenuation Relationships for Target Spectra	Magnitude, M	Rupture Distance, km	Focal Depth, km	Peak Bedrock Acceleration, g	Average Peak Bedrock Acceleration, g
Local Crustal	Campbell and Bozorgnia, 2008	6.2	1	NA	0.43	0.45
	Chiou and Youngs, 2008	6.2	1	NA	0.52	
	Boore and Atkinson, 2008	6.2	1	NA	0.40	

(1) Relationship by Zhao et al. (2006) limited to magnitude 8.5.

Probabilistic Considerations

The probability of an earthquake of a specific magnitude occurring at a given location is commonly expressed by its return period, i.e., the average length of time between successive occurrences of an earthquake of that size or larger at that location. The return period of a design earthquake is calculated once a project design life and some measure of the acceptable risk that the design earthquake might occur or be exceeded are specified. These expected earthquake recurrences are expressed as a probability of exceedance during a given time period or design life. Historically, building codes have adopted an acceptable risk level by identifying ground acceleration values that meet or exceed a 10% probability of exceedance in 50 years, which corresponds to an earthquake with an expected recurrence interval of 475 years. Previous versions of the IBC developed response spectra based on ground motions associated with the Maximum Considered Earthquake (MCE), which is generally defined as a probabilistic earthquake with a 2% probability of exceedance in 50 years (return period of about 2,500 years) except where subject to deterministic limitations (Leyendecker et al., 2000).

The recent 2012 IBC develops response spectra using a Risk-Targeted Maximum Considered Earthquake (MCE_R), which is defined as the response spectrum that is expected to achieve a 1% probability of building collapse within a 50-year period. The design-level response spectrum is calculated as two-thirds of the MCE_R ground motions. Since the MCE_R earthquake ground motions were developed by the USGS to incorporate the targeted 1% in 50 years risk of structural collapse based upon a generic structural fragility, they are different than the ground motions associated with the traditional MCE. Although site response is evaluated based on the MCE_R, it should be noted that seismic hazards, such as liquefaction and soil strength loss, are evaluated using the Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration (PGA), which is more consistent with the traditional MCE.

The 2012 IBC design methodology uses two mapped spectral acceleration parameters, S_S and S_1, corresponding to periods of 0.2 and 1.0 second, to develop the MCE_R earthquake. The S_S and S_1 coefficients for the site located at the approximate latitude and longitude coordinates of 45.37°N and 122.63°W are 0.95 and 0.41 g, respectively.

Estimated Site Response

The effect of a specific seismic event on the site is related to 1) the type and quantity of seismic energy delivered to the bedrock beneath the site by the earthquake and 2) the type and thickness of soil overlying the bedrock at the site. Ground motion hazard analysis was completed to estimate this site-specific behavior in accordance with Section 21.2 of ASCE 7-10. The ground motion hazard analysis consisted of three significant components: 1) estimation of ground surface response using recently developed attenuation relationships that are capable of modeling soil site conditions (deterministic evaluation), 2) estimation of ground surface response using code-based adjustment factors based on soil site class (probabilistic evaluation), and 3) comparison of the deterministic and probabilistic ground surface response spectra to recommend a site-specific response spectrum for design. The following paragraphs describe the details of the ground motion hazard analysis.

To estimate the deterministic ground surface response spectrum, recently developed attenuation relationships were used to evaluate amplification and/or attenuation of bedrock ground motions through the soil column at the site. Based on our review of the USGS deaggregation for the site (USGS, 2014), an event on the CSZ and crustal seismicity represent the largest contributing sources to the seismic hazard at the site. Considering this, we have chosen to estimate the deterministic ground surface response using 84th percentile ground motions from the following two earthquake scenarios: 1) a M9.0 subduction zone earthquake at a distance of 100 km from the site, and 2) a M6.2 crustal earthquake at a distance of 1 km from the site. The attenuation relationship of Youngs et al. (1997) and the recently developed BC Hydro relationship of Abrahamson et al. (2012) were used to evaluate the subduction zone earthquake response. The NGA ground motion relations published by Boore and Atkinson (2008), Campbell and Bozorgnia (2008), and Chiou and Youngs (2008) were used to evaluate the crustal earthquake response. One input parameter for the attenuation relationships is the average shear wave velocity in the upper 100 ft of the soil profile. Based on published correlations with standardized field data and our experience with similar subsurface conditions, we estimate the average shear wave velocity at the site is on the order of 1,100 ft/s. The resulting deterministic MCE_R ground surface response spectra are shown on Figure 6C. As required by Section 21.2.2 of ASCE 7-10, Figure 6C also shows the deterministic lower limit MCE_R spectrum. The deterministic MCE_R ground surface spectrum is taken as the larger of the 84th percentile ground motions and the deterministic lower limit. To estimate the probabilistic ground surface response spectrum, adjustment factors based on observed soil conditions are used to evaluate amplification and/or attenuation of bedrock ground motions through the soil column at the site. The site is classified as Site Class D, or a stiff soil site, based on the estimated average shear wave velocity in the upper 100 ft of the soil profile in accordance with Section 20.3 of ASCE 7-10. Corresponding short- and long-period adjustment factors F_a and F_v, of 1.12 and 1.59, respectively, were used to develop the probabilistic Site Class D MCE_R response spectrum shown on Figure 7C.

In accordance with Section 21.2.3 of ASCE 7-10, the site-specific ground surface MCE_R response spectrum is taken as the lesser of the probabilistic and deterministic MCE_R ground motions. Figure 7C shows a comparison of the deterministic and probabilistic MCE_R ground motions and indicates the code-based probabilistic Site Class D MCE_R response spectrum is appropriate for the site. The design-level response spectrum is calculated as two-thirds of the MCE_R response spectrum. We recommend using the Site Class D design response spectrum shown on Figure 8C for design of the reservoir.

Seismic Hazards

Liquefaction. Liquefaction is a process by which loose, saturated, granular materials, such as sand, and to a somewhat lesser degree soft, non-plastic silts, temporarily lose strength during and immediately after a seismic event. Liquefaction occurs as seismic shear stresses propagate through a saturated soil and distort the soil structure causing loosely packed groups of particles to contract or collapse. If drainage is impeded and cannot occur quickly, the collapsing soil structure increases the porewater pressure between the soil grains. If the porewater pressure increases to a level approaching the weight of the overlying soil, the granular layer temporarily behaves as a viscous liquid rather than a solid. As strength is lost, there is an increased risk of settlement, lateral spread, and/or slope instability. Liquefaction-induced settlement occurs as the elevated porewater pressures dissipate and the soil consolidates after the earthquake.

Based on preliminary evaluations, there is some risk of seismically induced soil strength loss in isolated soft layer(s) within the decomposed basalt that were encountered in some of the explorations at depths of about 20 to 40 ft below the ground surface. In our opinion, the risk of significant settlement due to seismically induced soil strength loss in these isolated zones is low. However, there is some risk of seismic slope instability at the site, and the presence of these loose and soft soil zones may increase the risk of slope movement during and immediately following an earthquake. We anticipate a ground improvement program will be completed at the site to limit the risk of seismically induced soil strength loss and slope instability.

Other Hazards. The risk of damage by tsunami and/or seiche at the site is absent due to the elevation of the site. In our opinion, the risk of liquefaction-induced lateral spreading and ground deformation at the site is low. As previously discussed, the surface trace of the Bolton Fault is located about 900 ft northeast of the site. Unless occurring on a previously unmapped or unknown fault, it is our opinion the risk of ground rupture at the site is low.

Based on our slope stability analyses completed for the project, there is a risk of seismically induced slope instability at the site associated with a relatively horizontal to shallow dip of soft layer(s) within the decomposed basalt. Soft layers were encountered locally in the borings between depths of about 20 and 40 ft below the ground surface. Our analyses indicate the potential seismic instability at the site would most likely consist of near-horizontal, translational block failures beneath the tank and on the sloping ground north of the tank. As currently planned, a ground improvement program will be completed beneath the tank footprint to reduce the risk of seismic movements beneath the tank. In addition, the top of the slope along the north side of the site will be flattened to decrease the risk of slope movement on the reservoir.

Conclusions

The 2012 IBC design methodology uses two spectral response coefficients, S_S and S_1, corresponding to periods of 0.2 and 1.0 second, to develop the MCE_R response spectrum. The S_S and S_1 coefficients for the site are 0.95 and 0.41 g, respectively. The results of the ground motion hazard analysis indicate the 2012 IBC Site Class D spectrum provides an appropriate estimate of the spectral accelerations at the site. We recommend using the Site Class D design spectrum shown on Figure 8C for the project.

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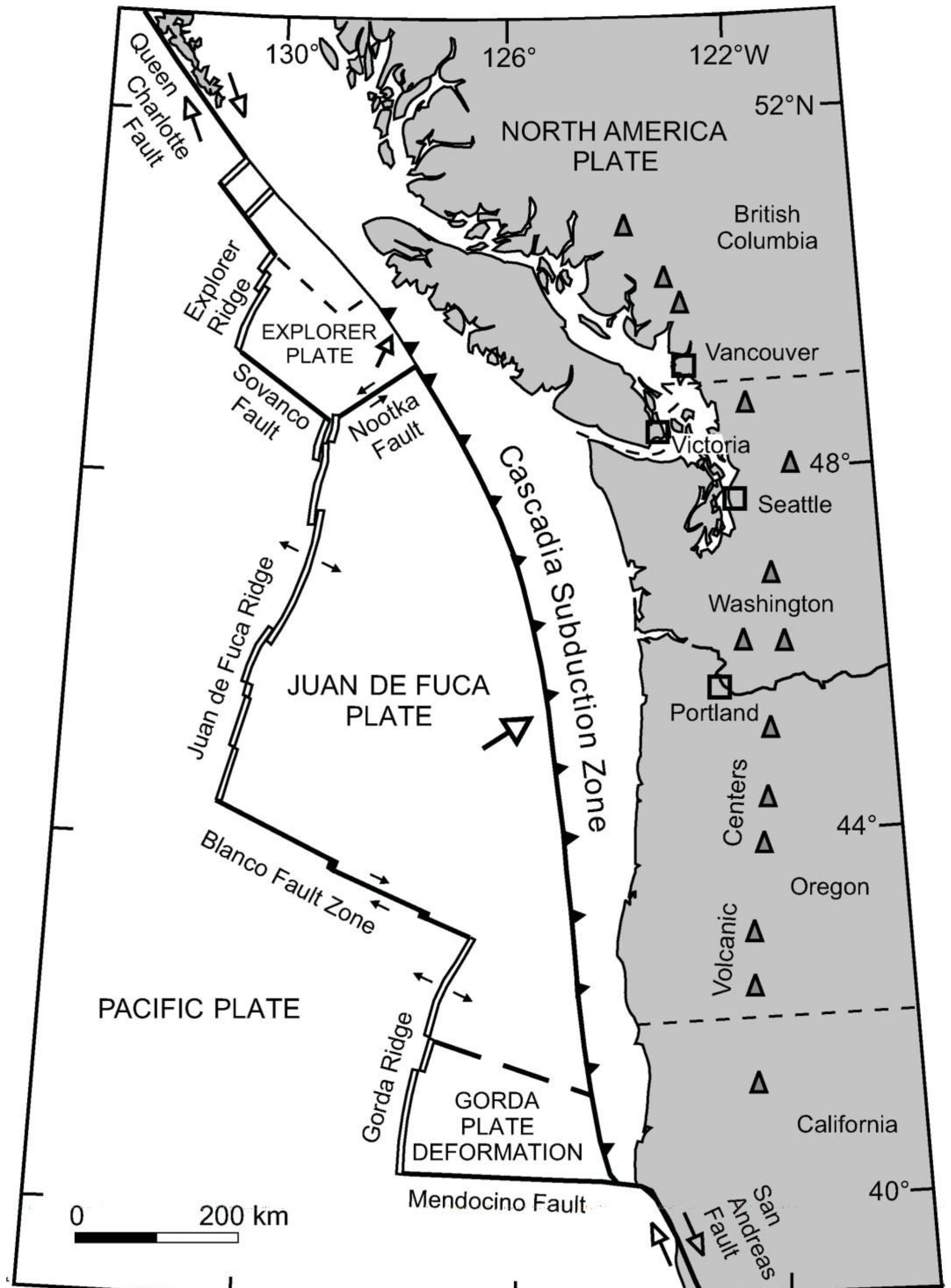
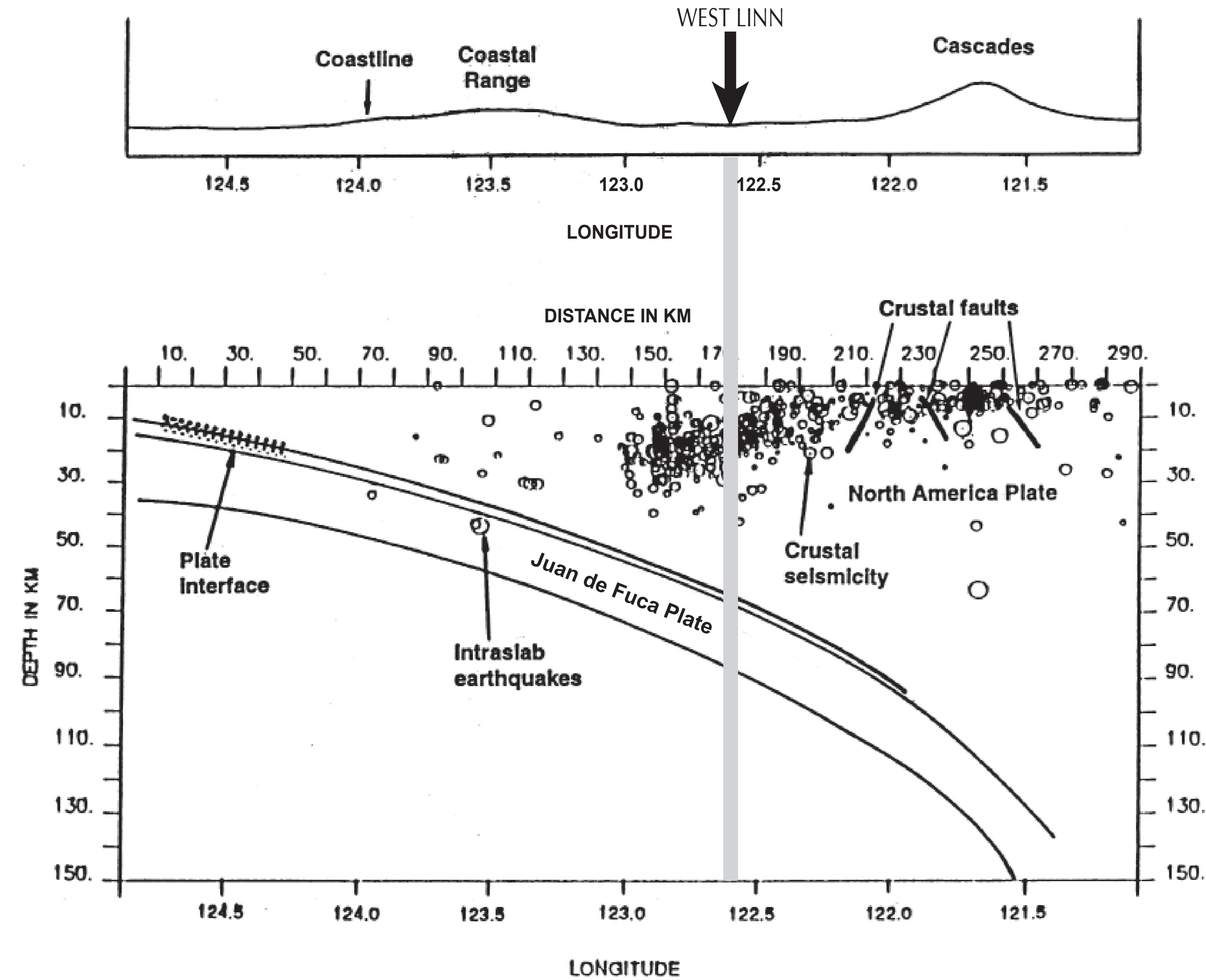


A) TECTONIC MAP OF PACIFIC NORTHWEST, SHOWING ORIENTATION AND EXTENT OF CASCADIA SUBDUCTION ZONE (MODIFIED FROM DRAGERT AND OTHERS, 1994)



B) EAST-WEST CROSS-SECTION THROUGH WESTERN OREGON AT THE LATITUDE OF PORTLAND, SHOWING THE SEISMIC SOURCES CONSIDERED IN THE SITE-SPECIFIC SEISMIC HAZARD STUDY (MODIFIED FROM GEOMATRIX, 1995)

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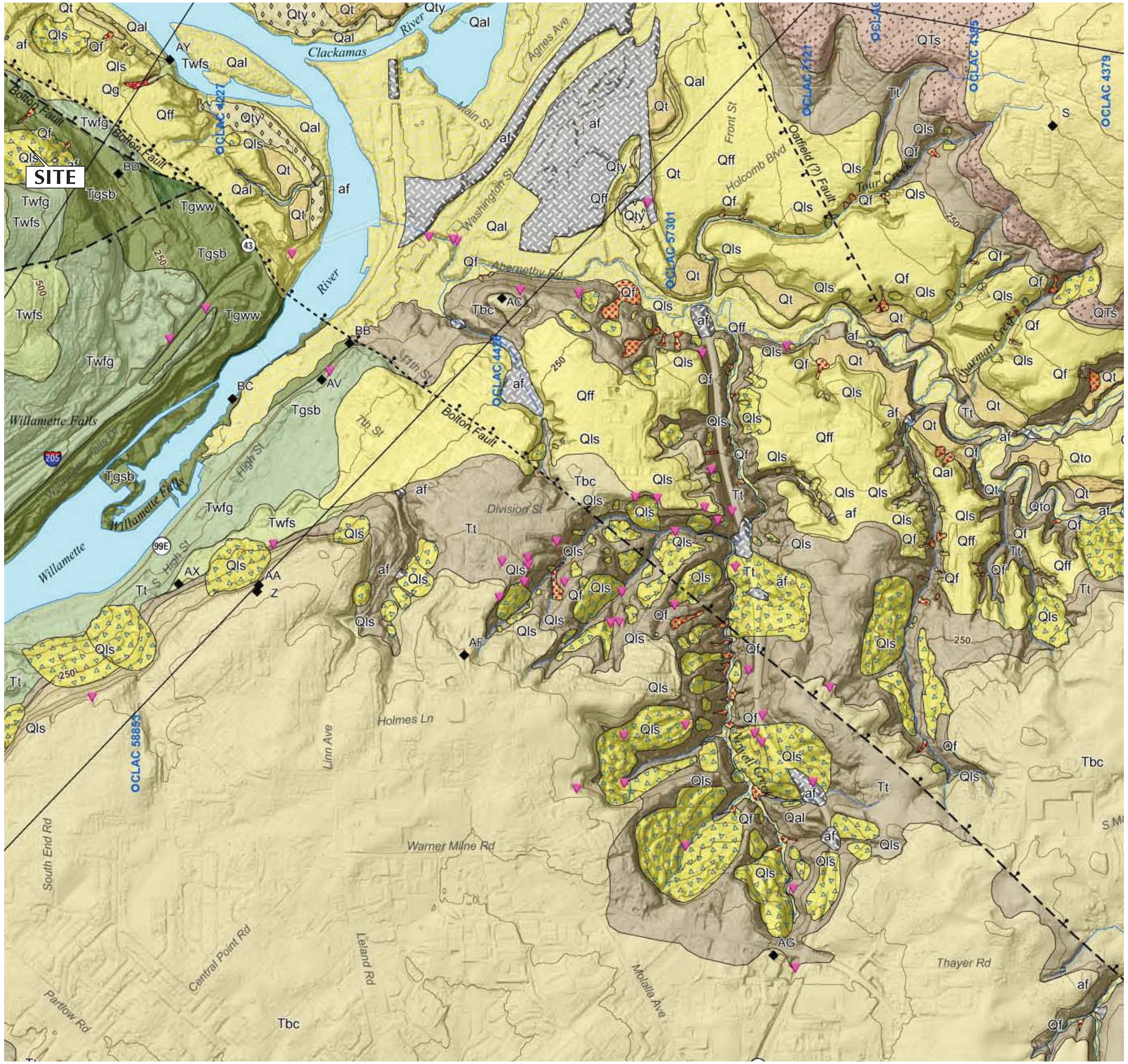
TECTONIC SETTING SUMMARY

JAN. 2015 JOB NO. 5338-A FIG. 1C



FROM: MADIN, I.P., 2009, GEOLOGIC MAP OF THE OREGON CITY 7.5' QUADRANGLE, CLACKAMAS COUNTY, OREGON: OREGON DEPARTMENT OF GEOLOGY AND MINERAL INDUSTRIES GEOLOGIC MAP SERIES 119.

MAP SYMBOLS

- Contact, approximately located
- Normal fault, approximate location
- Normal fault, concealed location
- Normal fault, inferred location
- A—A' Cross section line
- Water body
- ◆ Geochemical sample site, labeled with map code
- Location of water well used to construct cross section, labeled with Oregon Water Resources Department log identification number
- Location of minor debris flow from 1996-1997 storms (Hofmeister, 2000)
- Volcanic vent

EXPLANATION OF MAP UNITS

af — **Artificial fill** — Man-made deposits of mixed clay, silt, sand, gravel, and debris and rubble.

Quaternary Surficial Deposits

Qal — **Alluvial deposits (Quaternary)** — Gravel, sand, silt, and clay deposited in the active channels and floodplains of rivers and streams.

Qty — **Terrace deposits, younger (Quaternary)** — Lowest silt and sand (?) deposits capping strath terraces inset into Missoula Flood deposits along Abernethy Creek and the Willamette River near its confluence with the Clackamas River.

Qt — **Terrace deposits (Quaternary)** — Intermediate-elevation silt and sand (?) deposits capping strath terraces inset into Missoula Flood deposits along Abernethy Creek and the Willamette River near its confluence with the Clackamas River.

Qff — **Missoula (Bretz) flood deposits (Quaternary)** — Silt, sand, and minor gravel deposited by floods caused by the repeated failure of the glacial ice dam that impounded glacial Lake Missoula.

Qls — **Landslides (Quaternary)** — Chaotically mixed and deformed masses of rock, colluvium, and soil that have moved downslope in one or more events.

Miocene-Pleistocene fluvial sedimentary rocks

Qg — **Conglomerate (Pleistocene?)** — Pebble to cobble conglomerate exposed in the walls of a small stream canyon in the extreme northwest corner of the map area.

Tt — **Troutdale Formation sandstone, siltstone and mudstone (Miocene-Pliocene)** — Mudstone, claystone, sandstone, and minor conglomerate and tuff.

Miocene Columbia River Basalt Group lavas

Twfs — **Wanapum Basalt, Frenchman Springs Member, basalt of Sand Hollow (Miocene)** — Black medium-grained basalt flows with sparse plagioclase phenocrysts, well developed columnar jointing.

Twfg — **Wanapum Basalt, Frenchman Springs Member, basalt of Gingko (Miocene)** — Black medium-grained basalt flows with abundant plagioclase phenocrysts, well developed columnar jointing.

Tgww — **Grande Ronde Formation, basalt of Winter Water (Miocene)** — Flow or flows of fine-grained basalt.

North

0 1/2 1 MILE

0 2,000 4,000 FT

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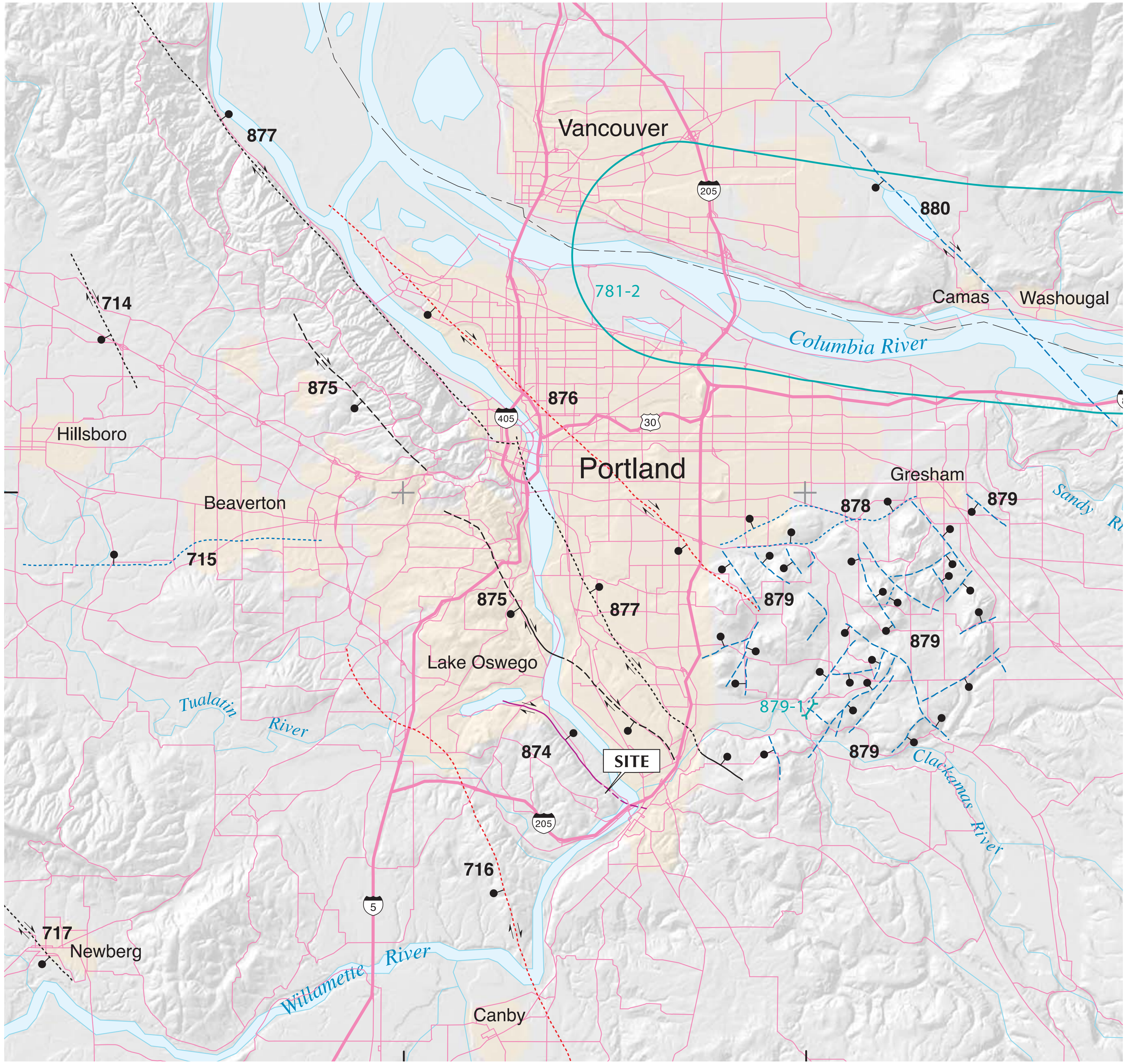
GEOLOGIC MAP

JAN. 2015 JOB NO. 5338-A FIG. 2C



MAP EXPLANATION

TIME OF MOST RECENT SURFACE RUPTURE

- Holocene (<10,000 years) or post last glaciation (<15,000 years; 15 ka); no historic ruptures in Oregon to date
- Late Quaternary (<130,000; post penultimate glaciation)
- Late and middle Quaternary (<750,000 years; 750 ka)
- Quaternary, undifferentiated (<1,600,000 years; <1.6 Ma)
- Class B structure (age or origin uncertain)

SLIP RATE

- >5 mm/year
- 1.0-5.0 mm/year
- 0.2-1.0 mm/year
- <0.2 mm/year

TRACE

- Mostly continuous at map scale
- Mostly discontinuous at map scale
- Inferred or concealed

STRUCTURE TYPE AND RELATED FEATURES

- Normal or high-angle reverse fault
- Strike-slip fault
- Thrust fault
- Anticlinal fold
- Synclinal fold
- Monoclinal fold
- Plunge direction of fold
- Fault section marker

DETAILED STUDY SITES

- 731-2 Trench site
- 781-2 Subduction zone study site

CULTURAL AND GEOGRAPHIC FEATURES

- Divided highway
- Primary or secondary road
- Permanent river or stream
- Intermittent river or stream
- Permanent or intermittent lake

FAULT NUMBER	NAME OF STRUCTURE
716	CANBY-MOLALLA FAULT
874	BOLTON FAULT
875	OATFIELD FAULT
877	PORTLAND HILLS FAULT
879	DAMASCUS-TICKLE CREEK FAULT ZONE

FROM: PERSONIUS, S.F., AND OTHERS, 2003, MAP OF QUATERNARY FAULTS AND FOLDS IN OREGON, USGS OPEN FILE REPORT OFR-03-095.

North

0 5 10 MILES

0 5 10 KILOME-

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LOCAL FAULT MAP

JAN. 2015 JOB NO. 5338-A FIG. 3C

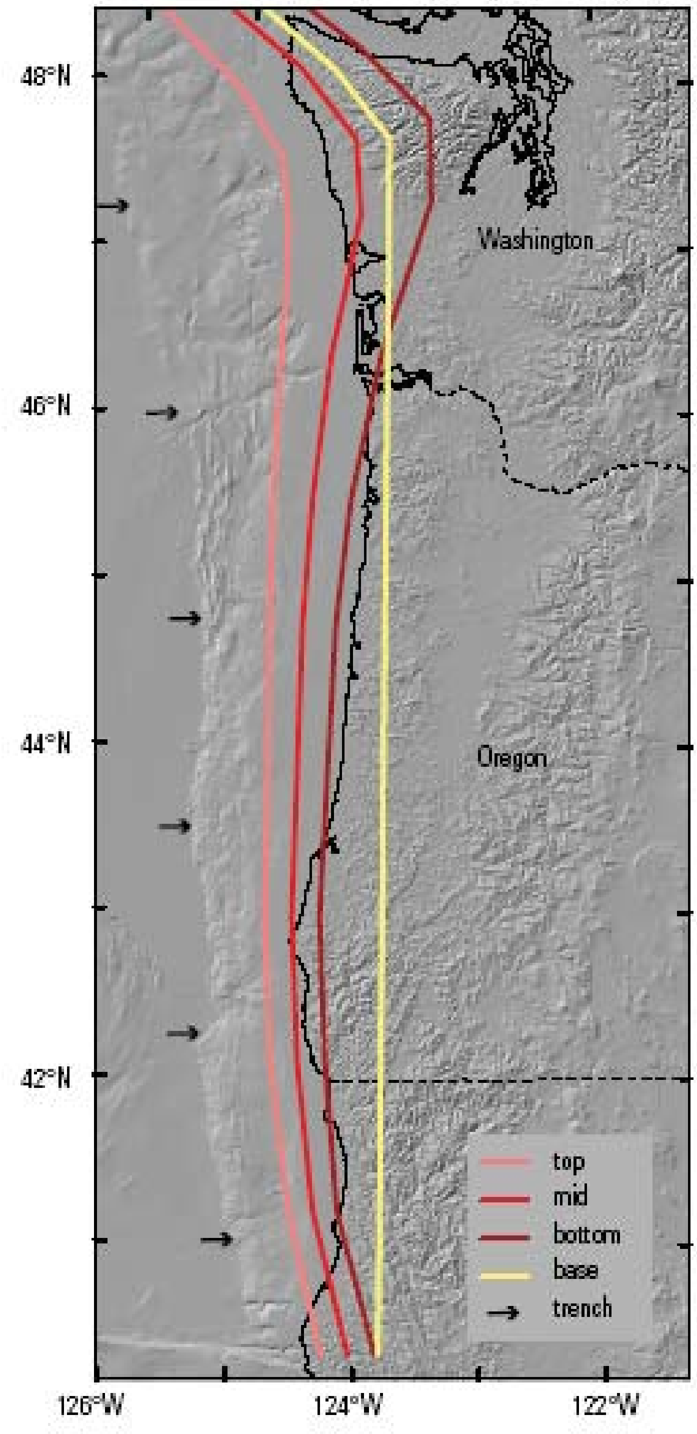

Figure 21. Location of the eastern edge of earthquake-rupture zones on the Cascadia subduction zone for the various models used in this study relative to the surficial expression of the trench: top, base of the elastic zone; mid, midpoint of the transition zone; bottom, base of the transition zones; base, base of the model that assumes ruptures extend to about 30-kilometers depth. Figure provided by Ray Weldon.

FROM: PETERSEN, MD, FRANKEL, AD, HARMSEN, SC, AND OTHERS, 2008, DOCUMENTATION FOR THE 2008 UPDATE OF THE UNITED STATES NATIONAL SEISMIC HAZARD MAPS: US GEOLOGICAL SURVEY, OPEN FILE REPORT 2008-1128

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ASSUMED RUPTURE LOCATIONS
(CASCADIA SUBDUCTION ZONE)

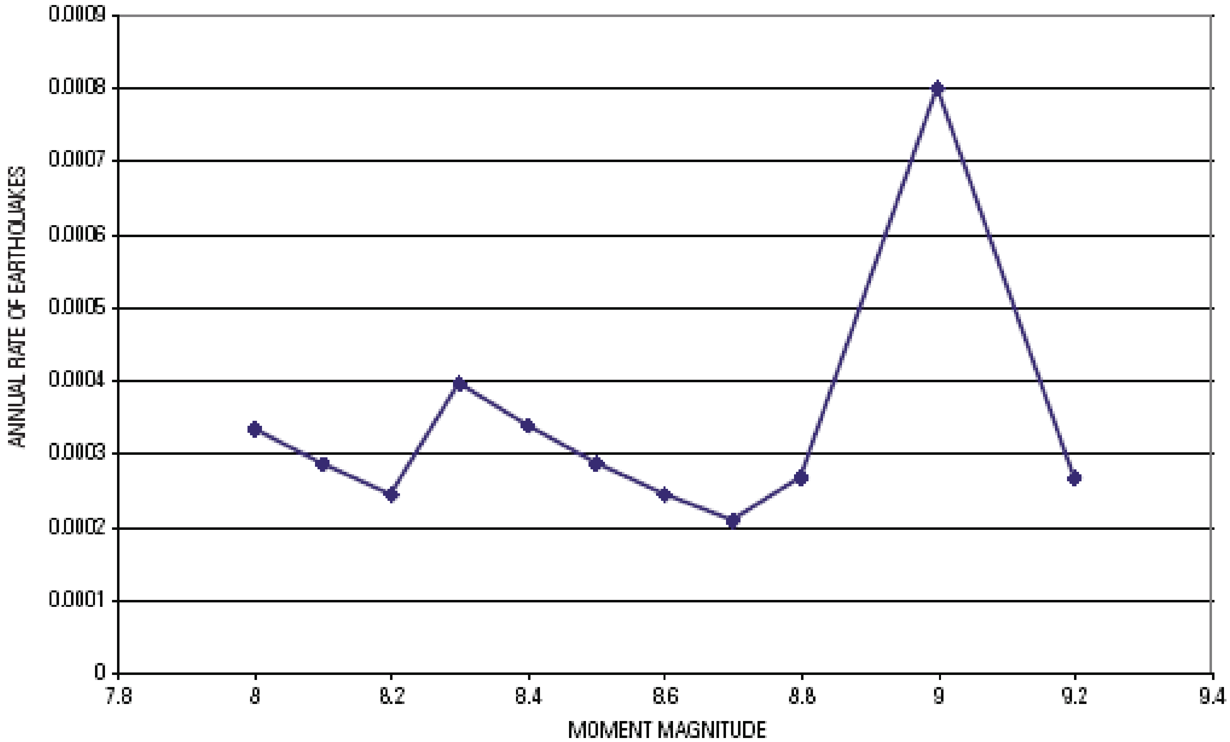

Figure 22. Magnitude-frequency distribution of the Cascadia subduction zone.

FROM: PETERSEN, M, FRANKEL, A, HARMSEN, S, AND OTHERS, 2008, DOCUMENTATION FOR THE 2008 UPDATE OF THE UNITED STATES NATIONAL SEISMIC HAZARD MAPS: US GEOLOGICAL SURVEY, OPEN FILE REPORT 2008-1128

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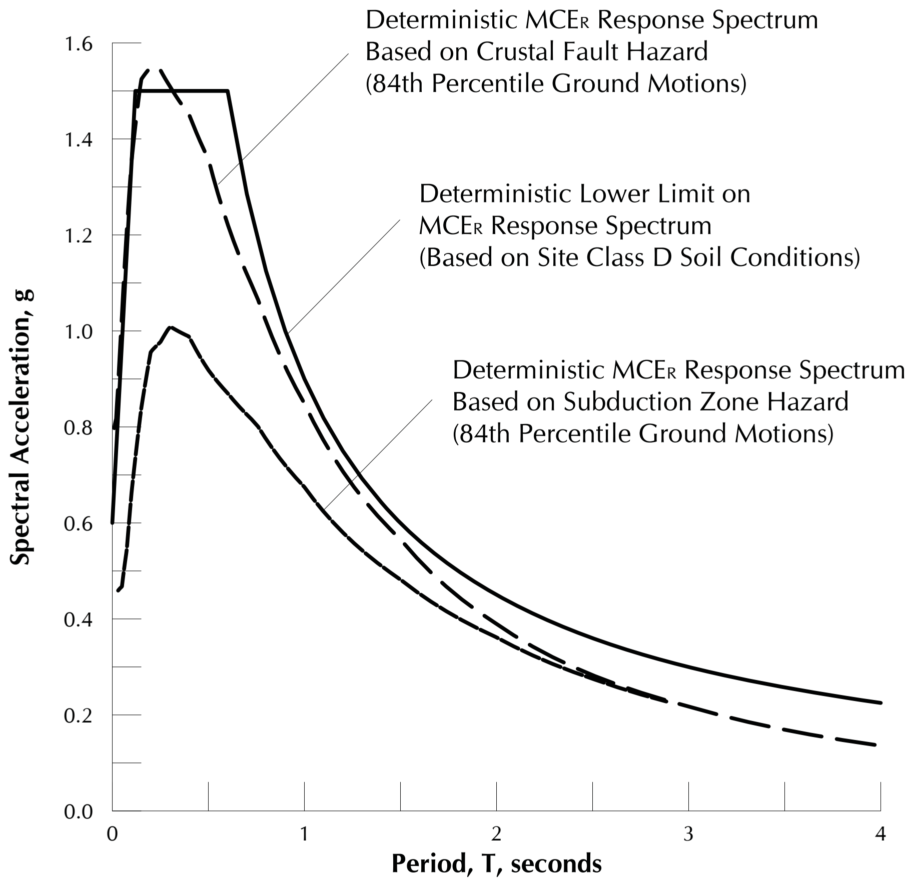
ASSUMED
MAGNITUDE-FREQUENCY DISTRIBUTION
(CASCADIA SUBDUCTION ZONE)

JAN. 2015 JOB NO. 5338-A FIG. 5C



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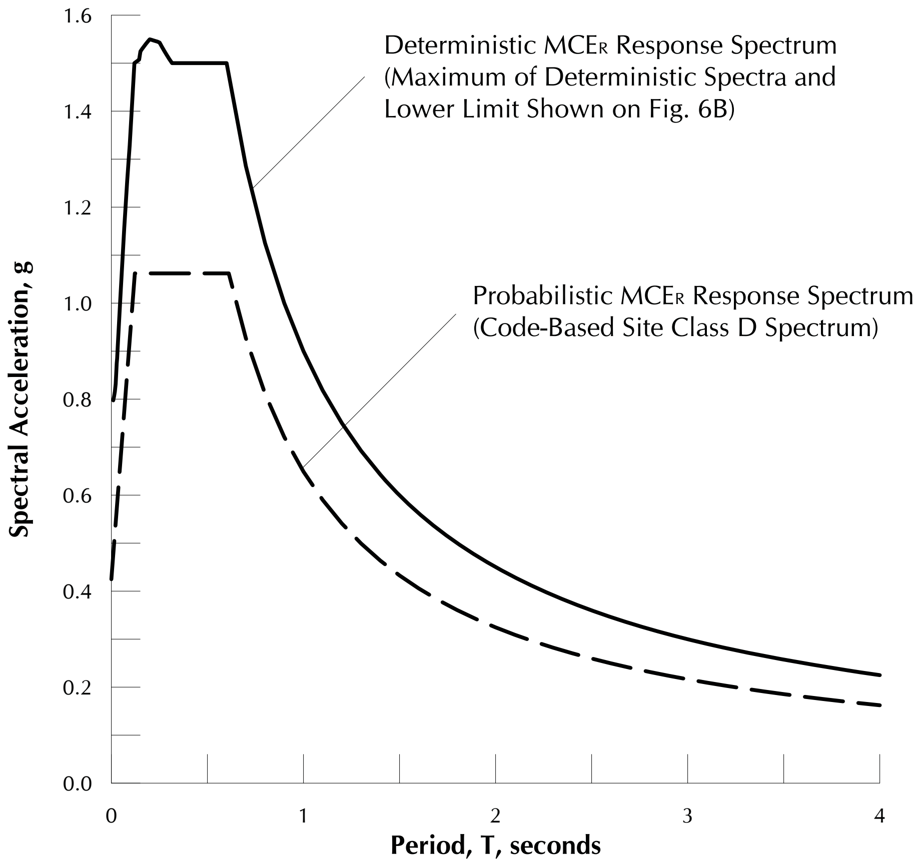
DETERMINISTIC MCER RESPONSE SPECTRA

(5% DAMPING)



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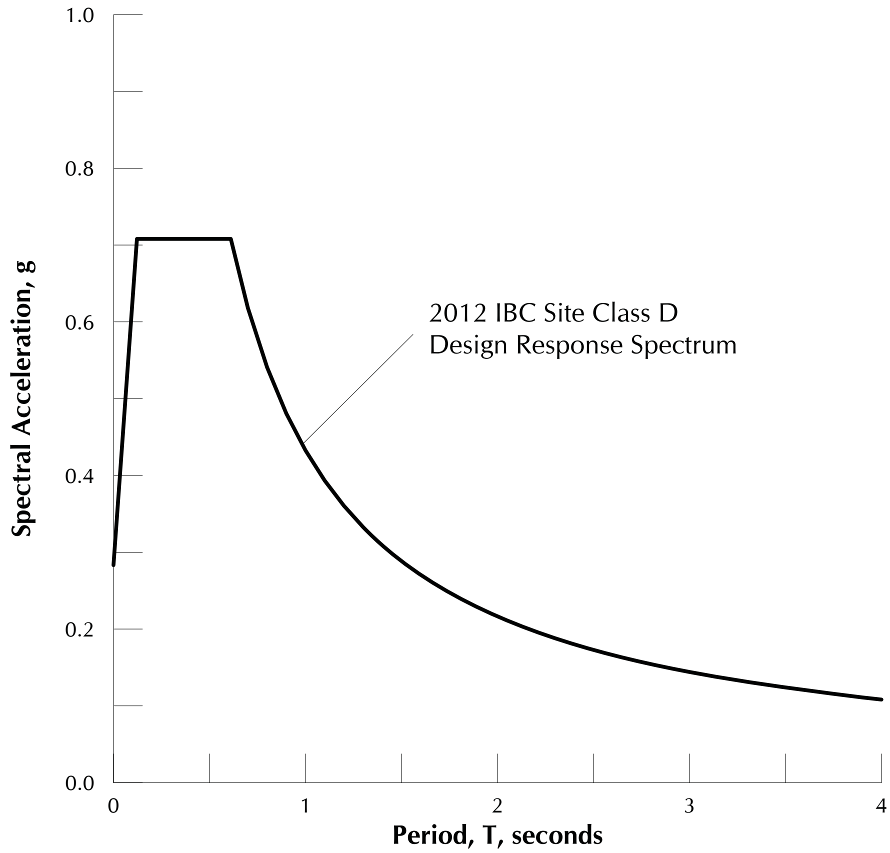
PROBABILISTIC AND DETERMINISTIC MCER
RESPONSE SPECTRA COMPARISON
(5% DAMPING)

JAN. 2015 | JOB NO. 5338-A | FIG. 7C



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DESIGN RESPONSE SPECTRUM
(5% DAMPING)

JAN. 2015 JOB NO. 5338-A FIG. 8C

APPENDIX D

Limitations

APPENDIX D

LIMITATIONS

This report has been prepared to aid the project team in the planning and design of this project. The scope is limited to the specific project and location described herein, and our description of the project represents our understanding of the significant aspects of the project relevant to the design and construction of the proposed reservoir.

The conclusions and recommendations submitted in this report are based on the data obtained from the explorations made at the locations indicated on Figure 2 and from other sources of information discussed in this report. In the performance of subsurface investigations, specific information is obtained at specific locations at specific times. However, it is acknowledged that variations in soil and rock conditions may exist between exploration locations. This report does not reflect any variations that may occur between these explorations. The nature and extent of variation may not become evident until construction. If, during construction, subsurface conditions different from those encountered in the explorations are observed or encountered, we should be advised at once so that we can observe and review these conditions and reconsider our recommendations where necessary.

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January 22, 2015 2381

Tom Boland, P.E.
Murray, Smith & Associates, Inc.
121 SW Salmon Street, Suite 900
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Geotechnical Engineering Services
Bolton Reservoir Seismic Landslide Evaluation
West Linn, Oregon

Dear Mr. Boland,

This letter summarizes our geologic reconnaissance and qualitative seismic stability evaluation of an ancient landslide surrounding the existing Bolton Reservoir. Recent geologic maps (Madin, 2009) have indicated the area including and surrounding the reservoir is an ancient landslide, but do not indicate if the landslide is currently active. Our evaluation was performed to identify any signs of landslide activity near the reservoir and to provide opinions on potential impacts of seismic landslide displacements on proposed improvements at the site.

Background Information

The Bolton Reservoir, shown on Figure 1, is located within a large ancient landslide mapped by the Oregon Department of Geology and Mineral Industries (DOGAMI). We understand that Murray, Smith & Associates, Inc. (MSA) has recently completed a Reservoir Siting Alternatives Analysis for the City of West Linn and has recommended that the City proceed with analyses to confirm the suitability of the existing reservoir site for new reservoir infrastructure. We also understand that MSA has been asked by the City to evaluate potential hazards posed by the landslide, including the potential for earthquake-induced movement.

Previous Landslide Studies in the Vicinity. In 1991 Landslide Technology (LT), a division of Cornforth Consultants, performed a geotechnical reconnaissance and design services for a landslide along Skyline Drive, approximately 600 feet southeast of the existing reservoir. The landslide affected two residential properties and a portion of Skyline Drive. In 1993, the landslide was mitigated with a rockfill buttress downslope of the two residences (LT, 1993). The buttress is keyed into basalt bedrock which underlies the landslide shear zone. In 2002, a waterline ruptured upslope of the landslide mitigation. LT performed a reconnaissance of the slide area to evaluate whether the water released by the pipe had an impact on the mitigation measures (LT, 2002). Surface cracking was observed near the centerline of Skyline Drive near where cracks were mapped in 1991. It was concluded that the rockfill buttress was performing adequately.

In 1998, LT performed a geotechnical investigation and developed mitigation measures for a shallow flow slide area near the northeast corner of the Bolton Reservoir site. The slide occurred in the 1970s, but additional ground cracks developed upslope of the slide in 1996 that suggested the slide was retrogressing (LT, 1998). The report indicated that the 1970s flow slide was likely due to a waterline break, and that the waterline had been relocated. As part of the 1998 study, one boring and several test pits were used to identify the limits of the flow slide.

In 2012 Geotechnical Resources Inc. (GRI) completed a geotechnical evaluation for the Bolton Reservoir conceptual siting analysis (GRI, 2012). As part of the study, GRI reviewed the landslide hazard maps prepared by DOGAMI and concluded that the Bolton Reservoir is located on a prehistoric (>150 years), deep-seated (>15 ft depth), translational landslide. GRI completed subsurface explorations to evaluate foundation conditions at the existing reservoir site. GRI's boring on the south side of the reservoir identified potential landslide debris to a depth of 60 feet. Readings collected on the inclinometer installed by LT in 1998 showed that 1/8 inch of shear displacement had occurred at a depth of 28 feet since installation.

Observations and Analysis

Reconnaissance. We visited the site on the November 10, 2014 to perform a reconnaissance of the ancient landslide and document any evidence of recent landslide activity. The general topography of the area is consistent with an ancient landslide, but surface expressions are muted or masked by development. General expressions of past movement include steepened slopes or scarps, translated slide blocks, and an offset creek channel. The landslide features shown on Figure 1 were mapped using a combination of field reconnaissance and interpretation from LiDAR maps prepared by DOGAMI. Figure 2 presents two schematic cross sections through the ancient landslide. The cross sections also show an interpreted potential shear zone that is consistent with surface expressions and limited subsurface data. The potential shear zone is shown for qualitative discussion, and does not represent known landslide conditions.

Walking reconnaissance was performed around the perimeter of the mapped landslide limits in an effort to locate signs and patterns consistent with recent landslide movement, including roadway cracking, buckling or separation of sidewalks, distorted driveway slabs, structural distress, and trees with curved trunks. Overall, the reconnaissance did not identify signs of distress consistent with active, on-going slide movement. Notably, sidewalks and pavement along Summit Street, Clark Street, and Linn Lane are in relatively good condition with little cracking or distortion. These streets cross the lateral limits of the mapped landslide where differential movements are generally most noticeable.

Several residences along Gloria Drive, near the headscarp of the landslide, show signs of structural distress to detached garages and raised decks. It appears that structures have experienced localized settlement and lateral displacement as evidenced by added bracing and leveling jacks supporting structural members. Detailed examination was not completed, but the type and location of the structural modifications are consistent with settlement or surficial slope movement.

Along Bridgeview Drive, approximately 500 feet from the intersection with Summit Street, a transverse crack was observed in the pavement. The crack appears to be an extension of cracks occurring in the sidewalk where there is separation between the sidewalk and a concrete retaining wall. The cracks do not exhibit signs of recent movement, and in our opinion, could be due to settlement or instability of localized cuts and fills.

Reconnaissance was also conducted along the northern limit of the mapped landslide, namely along Caufield Drive, West A Street, Hammerle Street, and Buck Street. Basalt outcrops were identified along Caufield Drive directly downslope of the existing reservoir. The outcrops were observed in the road cut as well as within the bed of the creek that runs just northeast of the existing reservoir. The basalt observed in these locations is consistent with basalt encountered in borings completed for the new reservoir and borings completed for landslide explorations.

Geology. The geology of the area is controlled by the following: (1) two members (Wanapum and Grande Ronde) of the Columbia River Basalt Group comprising the Tualatin Hills which are faulted by; (2) the Bolton Fault and associated splays forming the major northwest/southeast alignment of the Tualatin Hills; 3) an ancient terrace of the Willamette River covered with a mantle of Missoula Flood sediments; and 4) quaternary landslide features scattered along the front of the Tualatin Hills and the terrace above the Willamette River.

The Bolton Fault is mapped about 500 feet northeast of Bolton Reservoir, and extends in a northwest-southeast direction. The Oatfield and Portland Hills faults are mapped approximately 2.5 and 3 miles northeast of the site, respectively, and also trend in a northwest-southeast direction. These faults are included in the USGS Quaternary fault database (Personius, S.F., 2002) and are considered potentially active.

Also prominent in the vicinity of the site is the contact between the Wanapum and Grande Ronde members of the Columbia River Basalt Group. This contact is characterized by a well-developed, soil profile referred to as the Vantage Horizon. This feature formed during a long quiescent period that allowed soil to develop on top of the Grande Ronde basalt (17 to 15.6 million years old). The soil was subsequently covered by the Wanapum basalt (15.6 to 13 million years old). The Vantage Horizon is relatively weak compared to the surrounding rock and has acted as a landslide slip surface for multiple landslides in the Portland area.

Below the Bolton Reservoir site, there are a number of individual flow units of the Grande Ronde Basalt mapped (GMS-059, Beeson, 1989). These units were observed in the creek leading up from Caufield Street with some soil development apparent. Springs, with a combined estimated flow of 2 to 3 gallons per minute, were observed emanating from above basalt outcrops on Caufield Street.

Comments on Landslide Surrounding Bolton Reservoir. Based on our review of geologic maps and observations made during our site reconnaissance, we offer the following comments regarding the landslide surrounding the Bolton Reservoir site:

- In our opinion, the mapped landslide is a large, ancient, translational landslide mass consistent with other large landslides in the Portland area.

- Ancient translational landslides in the Pacific Northwest are typically marginally stable with factors-of-safety slightly above 1.0.
- The landslide surrounding the Bolton Reservoir has likely translated on a decomposed basaltic or tuffaceous, stiff clay layer. The borings indicate that the slide material is primarily stiff, clayey silt with SPT blow counts ranging between 10 and 20.
- As shown on the schematic cross sections, the basal shear zone is likely planar and dipping at a shallow angle to the northeast.
- We did not observe any signs of recent, deep-seated landslide movement during the site reconnaissance.
- There is a history of localized landslide movement at the reservoir site and the toe of the ancient slide, but the movements are likely related to a ruptured waterline and localized movements of steeper slopes at the toe of the ancient landslide.
- There have been no apparent operational difficulties at the existing reservoir due to deep-seated slide movements.

Seismic Behavior of Translational Landslides. Given the currently available data on the depth and extent of past landslide movement, it is not possible to perform a quantitative seismic stability evaluation at this time. To provide the City with information that would assist with Bolton Reservoir replacement project, the following paragraphs discuss on a qualitative basis the observed performance of large, translational landslides subjected to earthquake motions. Several case histories are cited that document seismically-induced landslide displacements in slide masses with characteristics similar to the landslide surrounding the Bolton Reservoir. The following section also discusses our experience regarding rate effects on shear strength of clayey soils at residual shear strength, and how strength gain could impact landslide displacements.

Seismic Deformation of Pre-Existing, Translational Landslides. During the past several decades, many active translational landslides have been subjected to earthquake-induced ground motions. Several case histories have been documented by earth scientists following large earthquake events. One of the more important observations gained from the post-seismic reconnaissance was that most pre-existing landslides have remained relatively stable with small to moderate displacements during the seismic event. Another observation is that translational landslides tend to move as coherent masses with small differential movements away from the slide margins. Several examples are listed below:

- The two largest historical earthquakes in the Pacific Northwest were the 1949 Olympia earthquake (M 7.1) and the 1965 earthquake near Seattle (M 6.5). During these earthquakes, it was observed that "seismic displacements associated with existing coherent slide blocks were typically less than 3 feet" (Chleborad, 1994).
- In 1976, a M 7.5 earthquake in Guatemala caused over 10,000 landslides. A USGS reconnaissance of the distribution and extent of landsliding indicated that, "despite strong

seismic shaking from the 1976 earthquake, pre-earthquake-landslide material mostly appeared to remain stable" (Harp, et al., 1981).

- In 1991, the M 7.0 Racha earthquake struck the Republic of Georgia triggering numerous landslides. Another USGS reconnaissance indicated that co-seismic displacements of active earth slides were generally less than 1 foot (Jibson, et al., 1994).
- The Loma Prieta earthquake was a M 7.1 event that struck the San Francisco Bay area on October 17, 1989. Shortly after the earthquake, engineers and earth scientists performed a reconnaissance of landslide and other geologic damage caused by the earthquake. These observations were summarized by the California Division of Mines and Geology (Manson, et al., 1992). Approximately 50 landslides were documented, and overall displacements were estimated based on fractures at the headscarp for each slide; 12 of these slides were active prior to the earthquake. The landslides had slope movements in the range of 1 to 12 inches at a distance of 10 kilometers from the epicenter.
- The 2010 Chile earthquake was a M 8.8 event that struck the central region and hilly coastal range. Earthquake reconnaissance by the Geo-Engineering Extreme Events Reconnaissance Association (GEER, 2010) documented that while scattered shallow slumps and rockfall were observed, deep-seated slides were relatively rare and in most cases the slides did not present a significant engineering issue. The report states that the most noteworthy aspect of deep-seated slides was their general absence. The scarcity of landslides is best illustrated by the fact that identifiable areas of prior extensive landslide activity had little or no evidence of seismically-triggered movement.

Dynamic Shear Strength. Many large, translational landslides involve movement along a relatively thin shear zone consisting of medium to highly plastic clayey soil. One of the key issues regarding the seismic behavior of ancient and active landslides is the response of clayey soils to earthquake loading conditions. There is limited data regarding the cyclic shear strength of stiff, clayey soils at residual strength. Several research studies on the shear properties of clays have focused on the effect of shearing velocity on shear strength (Lemos, Skempton and Vaughn, 1985: Kulhawy & Mayne, 1990, Meehan, et al., 2008). The trend of this data is that the undrained shear strength of clay increases approximately 5 to 10 percent per tenfold increase in the shearing velocity.

We have performed seismic stability evaluations for two landslides in the Portland area that have similar geology and geomorphology as the landslide mapped by DOGAMI at the Bolton Reservoir site. The landslides were both instrumented to determine the rate of movement occurring at the shear zone, and detailed studies were completed to quantify the strength gain that could be anticipated from the shear zone material during seismic shaking. Landslide instruments showed both slides moved at a rate on the order of 1/8 inch per year. By comparison, the average velocity induced by the various earthquake sources can reach up to 20 inches per second or more. Thus, the rate of shearing which could be generated by an earthquake is 8 to 9 orders of magnitude higher than the typical "creep" movement of reactivated ancient landslides. These prior studies concluded that the dynamic shear strength of the shear zone material could be significantly higher than the residual shear strength measured at low shear rates.

Finite element and Newmark modeling has also been completed on the two landslides mentioned above. The models incorporated strength gain during strong shaking to estimate displacements at several key positions along the shear zone. The models showed that earthquake-induced movements were typically between 1 and 3 feet in the main body of the slide, but movements up to 10 feet were calculated in localized areas. Without including dynamic shear strength into the models, displacements on the order of tens of feet would have been predicted, which is inconsistent with the observed behavior of translational landslides comprised predominantly of stiff soils.

Conclusions

Geotechnical reconnaissance of the ancient landslide around the Bolton Reservoir did not identify signs of active movement, especially along the margins where differential movement would be greatest. Occasional areas of suspect deformation were observed within the landslide mass, but in our opinion these are attributable to settlement, localized instability of cuts and/or fills, and shallow sloughing of steep slopes.

It is our understanding that GRI has performed local static and seismic stability analyses for local slope stability in the vicinity of the new reservoir and to address observations of past slope movement. Based on their analyses, GRI has recommended ground improvement, a setback from the existing bluff as well as regrading of the upper portion of the bluff to improve the local stability in the vicinity of the slide.

It is our experience that visual, walking reconnaissance is a useful tool to identify surficial landslide features. However, it is not as effective as instrumenting landslides with slope inclinometers and piezometers. Slope inclinometers can measure very small ground movements that may not be manifested at the ground surface, and piezometers can provide important groundwater head data acting on the shear zone.

Recent borings completed by GRI reportedly encountered the Vantage Horizon, which has been shown to be a weak zone that contributes to slope instability. In addition, soil layers between individual flows of the Grande Ronde basalt observed in the creek near Caufield Street could possess strength properties similar to the Vantage Horizon.

Case histories of existing landslides subjected to strong shaking indicate that translational landslides with stiff, clayey shear zones have typically undergone small to moderate deformations during earthquakes. In our opinion, the dynamic shear strength of the landslide shear zone is probably significantly higher than the static shear strength, which limits earthquake-induced movements. Finite element modeling of large, translational landslides in the Portland area has generally confirmed this observation. Based on our review, the ancient landslide at the Bolton Reservoir site is likely to move feet rather than tens of feet during a large earthquake.

Structures and piping can typically be designed to accommodate some level of landslide movement. Structures situated in the middle of large, translational landslides tend to "raft" along with landslide movement. In this event, differential settlements can occur and cause some structural cracking. Structures or utilities located on the margins of the landslide will be subjected to larger differential displacements than those located in the central portion of the slide mass. Alternative Site No. 2 identified in MSA's reservoir siting analysis is located along the scarp flank and would be expected to be subjected to larger differential displacements than the currently proposed location. Due to the larger differential displacements at Alternative Site No. 2, it is our understanding that the civil/structural design team prefers Site No. 1 adjacent to the existing reservoir.

We recommend that site-specific, quantitative stability analyses be completed if the civil/structural design team determines the magnitude of seismically-induced landslide movement is needed for design of the new reservoir. Geotechnical investigations and analyses to further evaluate the geotechnical feasibility of the site should include deep exploratory borings and instrumentation to characterize the conditions and geometry of the ancient landslide mass and determine if active movement is occurring.

We trust that this letter is sufficient for your current needs. If you have questions regarding the information presented above, please call Chris Carpenter at 503-452-1100.

Very truly yours,

CORNFORTH CONSULTANTS, INC.

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Senior Associate Geologist

Christopher I. Carpenter
Associate Engineer

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Enclosures: Figure 1 – Site Plan
Figure 2 – Cross Sections

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Limitations in the Use and Interpretation of this Geotechnical Report

Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally accepted engineering principles and practices. This warranty is in lieu of all other warranties, either expressed or implied.

The geotechnical report was prepared for the use of the Owner in the design of the subject facility and should be made available to potential contractors and/or the Contractor for information on factual data only. This report should not be used for contractual purposes as a warranty of interpreted subsurface conditions such as those indicated by the interpretive boring and test pit logs, cross-sections, or discussion of subsurface conditions contained herein.

The analyses, conclusions and recommendations contained in the report are based on site conditions as they presently exist and assume that the exploratory borings, test pits, and/or probes are representative of the subsurface conditions of the site. If, during construction, subsurface conditions are found which are significantly different from those observed in the exploratory borings and test pits, or assumed to exist in the excavations, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. If there is a substantial lapse of time between the submission of this report and the start of work at the site, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, this report should be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

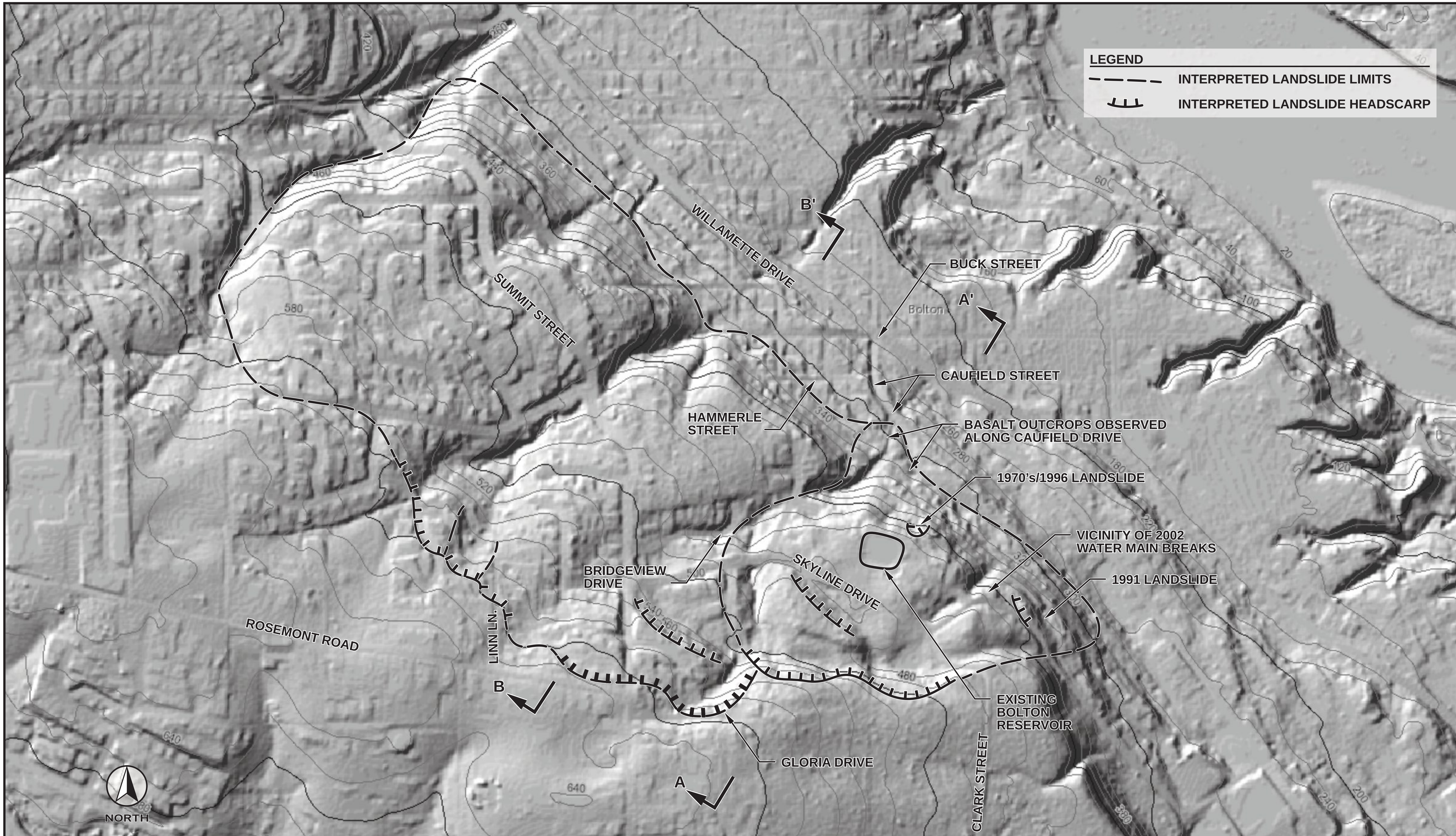
The Summary Boring Logs are our opinion of the subsurface conditions revealed by periodic sampling of the ground as the borings progressed. The soil descriptions and interfaces between strata are interpretive and actual changes may be gradual.

The boring logs and related information depict subsurface conditions only at these specific locations and at the particular time designated on the logs. Soil conditions at other locations may differ from conditions occurring at these boring locations. Also, the passage of time may result in a change in the soil conditions at these boring locations.

Groundwater levels often vary seasonally. Groundwater levels reported on the boring logs or in the body of the report are factual data only for the dates shown.

Unanticipated soil conditions are commonly encountered on construction sites and cannot be fully anticipated by merely taking soil samples, borings or test pits. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. It is recommended that the Owner consider providing a contingency fund to accommodate such potential extra costs.

This firm cannot be responsible for any deviation from the intent of this report including, but not restricted to, any changes to the scheduled time of construction, the nature of the project or the specific construction methods or means indicated in this report; nor can our firm be responsible for any construction activity on sites other than the specific site referred to in this report.



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NOTE: BASE MAP FROM OREGON DEPARTMENT OF GEOLOGY AND MINERAL INDUSTRIES LIDAR DATA VIEWER AND STATEWIDE LANDSLIDE INFORMATION LAYER FOR OREGON (SLIDO) (WWW.OREGONGEOLOGY.ORG/DOGAMILIDARVIEWER) (WWW.OREGONGEOLOGY.ORG/SLIDE/INDEX.HTML)

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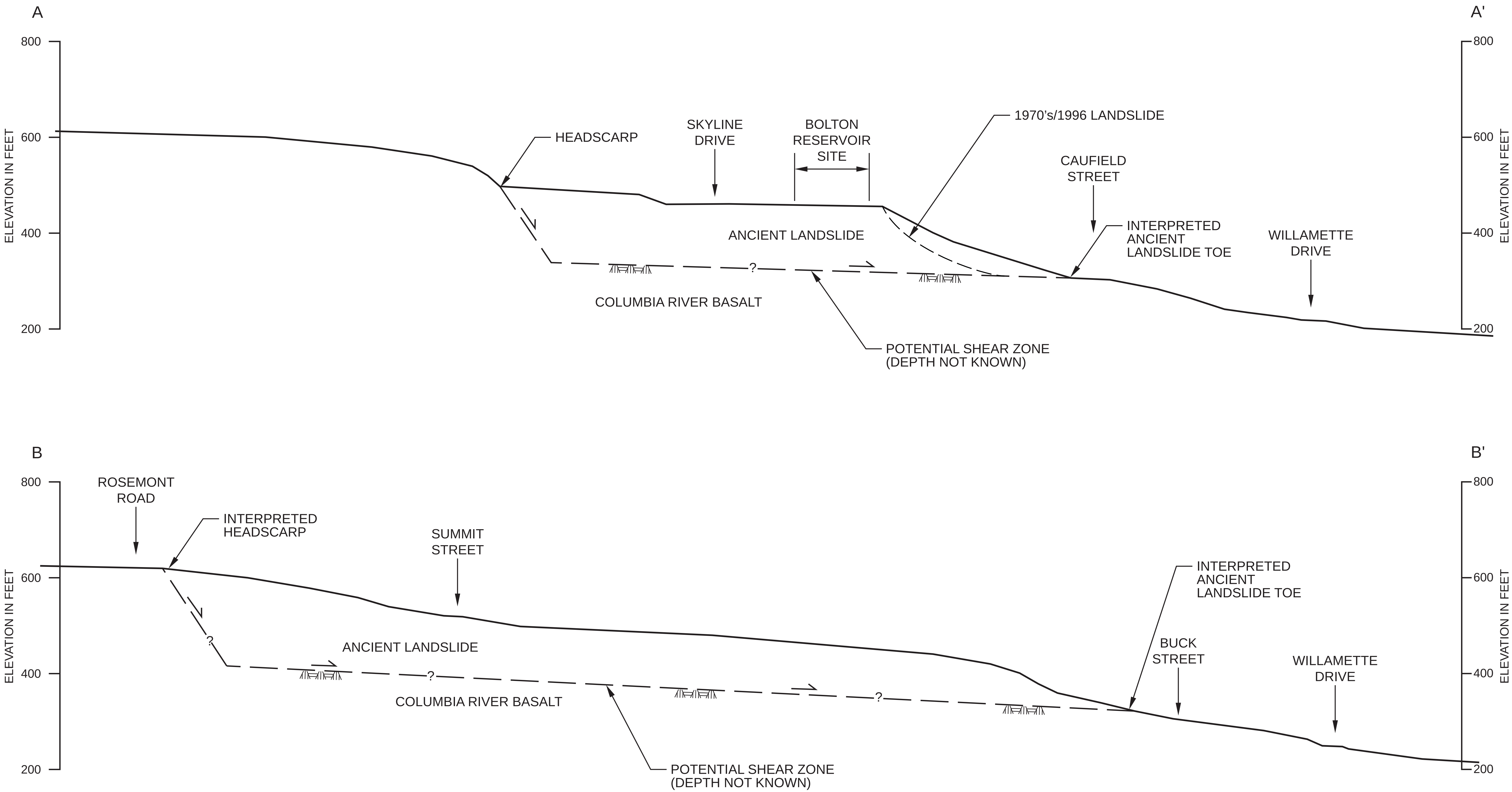
SITE PLAN

BOLTON RESERVOIR REPLACEMENT
WEST LINN, OREGON

JAN 2015

PROJ. 2381

FIG. 1



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SCHEMATIC CROSS SECTIONS

BOLTON RESERVOIR REPLACEMENT
WEST LINN, OREGON

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FIG. 2